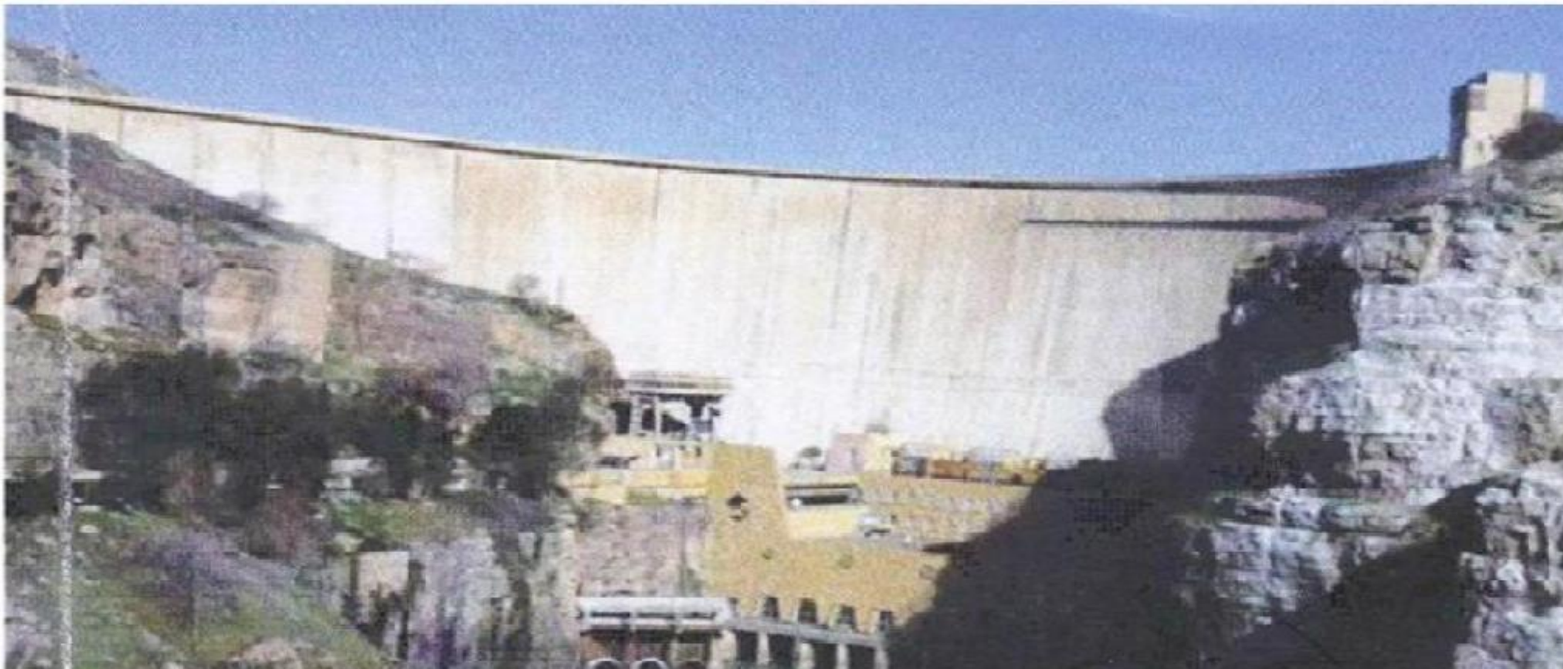

Dam Safety Review: General Principles and Procedures - Part 1

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Author's Note

This book has been prepared and published in two Parts as collection of separate papers on issues related to Dams Safety to contain as much information on this topic as possible. It provides a review of current practices on the subject together with actual accidents and failures due to natural causes, failure of designers to recognise proper safety requirements or operators' short sights in operation or maintenance . The author wishes to extend his warmest thanks to Mr. Varoujan Sissakian for the editing of the manuscripts of all the papers and to Professors Nadhir al- Ansari, Jan Laue and Sven Knutsson from Lulea University of Technology for the generous grant offered by the university for the publication.

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Part 1

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Dam Safety: General Considerations

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Abstract

Dams construction is an old art practiced by man since thousands of years. History of dams shows great innovations in this field, but failure cases, however, indicate gaps in human knowledge of safety measures that could have stopped such failures. Available statistics show of a great boom in building dams during the past century and indicate at the same time large number of failures associated with losses in human lives and material damage. Uses of these dams during this period, apart from flood control and storing water for irrigation were also for hydropower generation, navigation, drinking water supply, recreation and in mining operations as tailing dams. Reduced dam safety leading to failures, accidents and higher safety hazards were caused by insufficient knowledge of the geological conditions and in using wrong or deficient foundation treatment. Dam safety was compromised in cases of insufficient hydrological data and design of inadequate spillways. Misinterpretation of the seismic conditions of the area and adopting seismic criteria compatible with such seismic conditions is also added as one more reasons of failures. Human mistakes and errors have undermined safety in many cases in the operation of dams leading to grave safety issues including many failures. Safety hazards also were exasperated by increasing population and land use in the downstream areas of dams and by failing to do necessary inspection and maintenance or upgrading works. More emphasis over dam safety measures is needed now in our existing dams and in their future development of dams if they are to continue delivering their benefit without causing harm to human communities.

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1. The early beginnings

Dams are man-made structures that are primarily used to store and/or divert water. Other uses such as power generation and recreation were added to those uses in the last century. The benefits of dams were known to the ancients since thousands of years, and so we see that old civilizations had discovered these benefits and had built dams to harness large rivers and store water for the use, mainly in irrigation and produce the food they needed. Some other dams were built to divert water but to serve irrigation in the end also. Those people had used trial and error coupled with intuition in order to achieve this, but in many times met with failures and undoubtedly paying the price in human lives.

Recorded history and archeological findings show that the oldest dam in the world dates back to 3000 BC; this was the Jawa Dam in Jordan, which was actually the largest in a series of dams that were all parts of one reservoir system. This dam was 4.6m high, 24.4m long, with a base of 4.6 m, and It created the Jawa Reservoir that had a capacity of more than 30,000m^3. The dam itself was so well designed and constructed that the ancient structure stood till just few years ago, when it was partially ruined due to human intervention, Figure 1, [1].

Most ancient dams were simple gravity or earth fill dams, the Jawa Dam, however, was constructed from gravel reinforced with rock fill behind the upstream wall in order to protect the wall from the water pressure breach. This feature reflects the awareness of the ancient constructors of safety measures that could be incorporated in the design to protect the structure. In similar case, we may mention the Nimrud Dam which was built on the Tigris River in Mesopotamia, now Iraq, during the Chaldeans time of the New Babylonian era (330-612BC) which was meant to divert the flow from the Tigris River to the Grand Nahrawan Canal and it remained in service until its final collapse in the thirteen century. Sir William Willcocks, the late nineteen century British engineer spoke of this dam saying:

“The construction of the Grand Nimrud Dam and the Great Nahrawan Canal involved colossal volume of work and great deal of planning and surveying works. In considering the size of the “Grand Nimrud Dam’ we should remember that this dam had to be of such volume and workmanship as to resist Tigris floods which from hydrological calculation can reach up to 12,000 m^3/second. The dam continued to function for about three thousand years and its destruction and progressive abandonment lasted from the mid-10th century onwards mirroring the Abbasid Caliphate's decline”.

It is unfortunate that the remains of this dam are no longer in existence; being washed away by the Tigris River, which had changed its course. But from the words of Willcocks it may be inferred that, apart from the skills and workmanship used in its construction, the dam must have had many safety features in its design, [2].

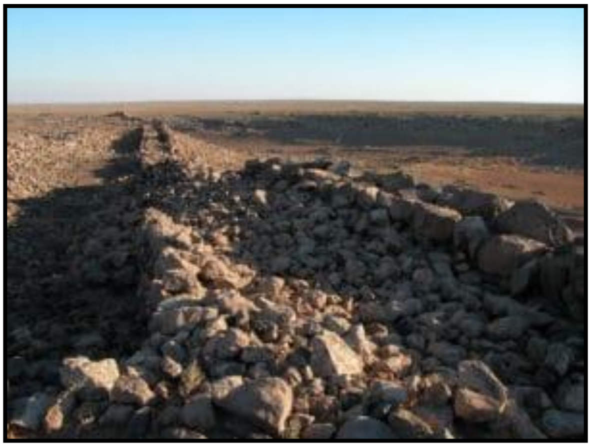

Figure 1: Remnants of Jawa Dam in Jordan [1].

History furnishes us with many cases of mixed success and failure; one such example is the case of Marib Dam in the kingdom of (Sheba) in Yemen, which was built around 750BC. In its good days, the dam supported a flourishing agriculture and prosperous communities. Notwithstanding the dam good construction, it had been overtopped several times during its history, but always had been repaired. In the recurrent overtopping, severe flooding had occurred and the dam proved to be a constant hazard to the communities living in the downstream and led to the abandonment of towns and cities. After the last failure, the inhabitants were forced to leave the area for their safety, and this caused serious mass migration to Syria and Mesopotamia. The final collapse of the dam occurred most probably in 575AD, after more than 750 years of service [2], [3]. Remnants of the dam are located about 150km east of Sana'a where a modern dam has been constructed lately, Figure 2, [4].

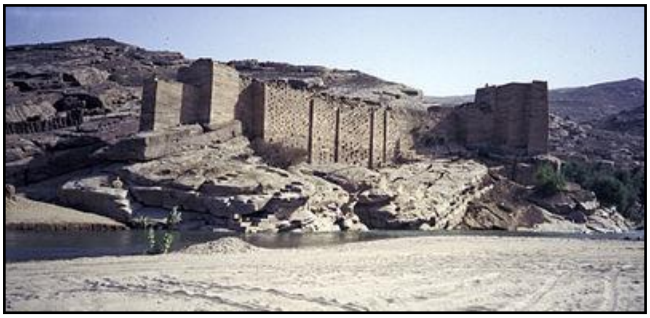

Figure 2a: Ruins of Marib Dam [4].

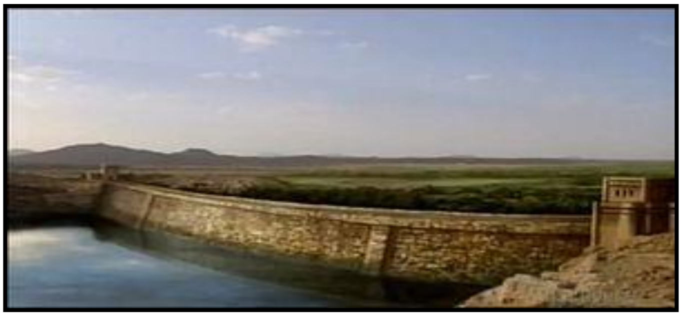

Figure 2b: Artist impression of the ancient Marib Dam [4].

Another example of failure is the case of Saad el-Kafara, which was masonry-embankment dam on Wadi al-Garawi 10km southeast of Helwan in Egypt. The dam was built in around 2650BC by the ancient Egyptians for flood control, and it was the oldest dam of such size in the world. The dam was 14m high, with a base width of 98m. It was originally 113m long, but now there are only the remains of construction on both sides of the wadi, and it is estimated that it was necessary to excavate and transport approximately 100,000m^3 of rock and rubble for its construction. No mortar was used in the dam: since ancient Egyptians did not use mortar as a cementing material. Considering the construction methods and technology available at that time, and considering the volume of gravel and rock fill that had to be transported from the wadi terraces to the dam's core and facing, the construction can be estimated to have taken 10-12 years. In cross-section, the Sadd el-Kafara dam consisted of three construction elements of a total width of 98m, which differ in composition and function: these were, i) An impervious central core of essentially calcareous silty sand and gravel, ii) Two sections of rock fill on either side (upstream and downstream) of the core which support and protect it. The fill consists of rocks, usually 30cm thick, but these also range in thickness (10-60cm). The quarried fill material was dumped down haphazardly, and the cavities between these rocks were not filled with gravel or debris, iii) upstream and downstream slope protections of blocks/ashlars placed in steps on the slopes of the rock fill, Figure 3, [5], [6]. The total volume of the reservoir when fully impounded to an elevation of 125(m.a.s.l) is about 620,000m^3. Below elevation 123.5(m.a.s.l) about 465,000m^3 could be stored. The purpose of the dam is still controversial, for such a large-scale reservoir may have been needed either to fulfill a heavy demand, such as for irrigation, or to protect the downstream area from flooding, but there is no evidence of cultivated land around the dam. Also, the absence of spillways in the dam indicates that the reservoir was not built for irrigation. On the other hand, geographical and geological conditions prevailing in the catchment area of the Wadi Garawi indicate that sudden and heavy rainfalls can lead to flash floods with disastrous effects in narrow valleys. Flood discharge can be estimated anywhere between 50 to 250m^3/sec, and even nowadays inhabitants in the region report the recent occurrence of floods several meters high, which have destroyed villages and claimed lives. So, in all probabilities the dam was intended for the retention of the rare but violent floods in the valley of wadi el Garawi.

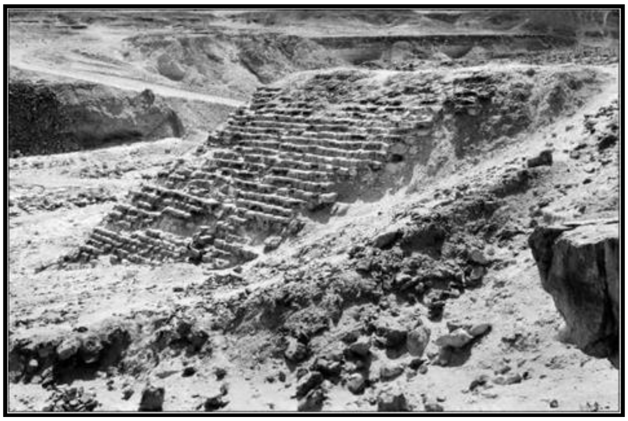

Figure 3: Remains of the Sadd el Kafara dam (about 2600BC), on the right bank of the wadi Garawi one of the oldest known large dam in history. View from upstream (photo of G.Albecht), [5].

Evidence shows that Sadd el-Kafara dam was never completed, as there are no signs of siltation in the reservoir. Most probably the dam was partially completed when a sudden high flood occurred and washed away the central part of the dam. There is evidence also that the upstream rock fill was almost (or fully) completed, but a gap still remained in the middle section of the downstream rock fill, and perhaps also in the core, when the flood occurred. The inadequately protected core could not withstand the gush of the water overtopping it and was carried away causing the central part of the dam to collapse. Modern assessments of the dam's stability lead to the conclusion that the design was basically correct, though conservative [6]. This example shows that the builders of the dam took a considerable risk for not providing an escape for the flood water during construction, which is something that has been taken care of in constructing modern dams. But even in some recent cases, however, serious failure of coffer dams during construction occurred showing underestimation of the floods adopted during diversion. In both cases this is due to hydrological miscalculation and bad judgment causing serious safety hazards.

Development of the skills for dams' construction grew from damming small streams to building diversion weirs needed for diverting irrigation water by the use of materials that were available at hand. In ancient Sumer, in Mesopotamia, 3000BC, people placed rolls of mats woven from palm trees fronds and impregnated with bitumen in the stream and then dumping earth behind them to create the required barrier. Later on, such weirs were built from burnt clay brick masonry cemented with various types of mortars, and proper gravity dams exceeding 15m in height were built also from cut stones masonry for river diversion, irrigation, and flood control. As examples to be noted are the old Diyala dam and the old Adhaim dam, which were most probably built by the Chaldean king of new Babylonia, Nebuchadnezzar II (605–562BC) which bear witness, and they were complementary to the works of the Great Nahrawan canal. The two dams continued to function until their collapse, due to lack of maintenance and conflicts, in 912AD and 1150AD respectively. Trial and error and the accumulation of experience helped in the building of larger dams by the use of different materials and for many more uses other than only irrigation.

Of all the dams that were built during many thousands of years, many of them can be described as being unsafe dams. We have to admit that history is silent on catastrophes related to loss of lives, which occurred from dam failures, but we can assume that such losses occurred. The major weakness of those dams was their incapacity to withstanding high floods due to inadequate knowledge of hydrology, or weak foundation due to misjudgment of the foundation conditions or maybe being subjected to seismic events. Some of the dams which had passed these tests collapsed finally after long years of service due to aging. Aging remains up to this moment as a major cause of failure, and therefore many old dams, which are still in operation may pose safety hazards to communities living downstream of them and require special thought for decommissioning them.

2. Evolution of Dams' Design and Construction

While construction of dams continued for ages, development in the design and types of these dams evolved. The need for building more dams increased with the development of human societies and their increased use of water and enhanced knowledge of hydraulics. In this development the Romans may be regarded as pioneers for their advances in hydraulic engineering, which led them to build higher dams and develop new types. They were creative and abundant in dam construction, and during the height of the empire, they built large number of gravity dams, most notably the Subiaco Dams.

The Subiaco Dams were a cascade of three gravity dams on the Aniene River in Subiaco, Italy, that were built during the reign of Nero (54–68AD). The largest of these dams stood 165 feet tall and was the tallest dam in the world until its destruction in 1305; which was due to mismanagement, [7]. The Romans also constructed the world's first arch dam in the province of Gallia-Narbonensis, now modern-day southwest France, in the first century BC. The remains of the Glanum Dam, the first recorded true arch dam in history, were discovered in 1763. Unfortunately, a modern arched gravity dam replaced the ancient structure in 1891, and all remnants of the Glanum Dam were lost. The Romans were also responsible for constructing the world's first buttress dam. Many such dams were built in the Iberian Peninsula, although they tended to fail due to their too-thin construction. As a consequence of the industrial revolution and the increased demand for water during the modern times, large dams began to appear especially after the introduction of concrete in their construction. Major advances in concrete dam design were made from 1853 to 1910 by British and French engineers. During this time, understanding of the relationship between the precise weight and profile of gravity dams and the horizontal thrust of water increased extensively. In 1910, further advances were made as engineers began to take a more three dimensional approach to dam engineering, examining the effect of individual stresses and deflections on multiple points rather than on the structure as a whole.

By recognizing the complexity of the structure and understanding its interconnectedness, engineers were able to make exponential advances in dam engineering. The world's largest and most complex dams have all been built within the last century due to engineering as well as technological advances. In addition to supplying water and controlling flooding, modern dams are often constructed to provide hydroelectric power. Striking examples of advanced engineering are found in such dams as Hoover Dam, a concrete arch-gravity dam constructed on the Colorado River in 1936. The massive dam, which impounds Lake Mead, is 726 feet (224 m) tall and has a reservoir capacity of 28,537,000 acre feet (> 35 billion m^3). Hoover Dam however lost its title as the highest in the world to other dam's long time ago.

3. Present day Statistics on Dams

The numbers of all dams in the world has exceeded 800,000 dams in 2007, out of this about 40,000 were large dams [8]. The register of the International Commission on Large Dams (ICOLD) indicates today a dramatic increase within the past thirteen years to about 60,000 large dams, Figure 4 [9]. Many of these dams serve multipurpose while other serve a single purpose as shown in Table 1. The first twenty highest dams in the world today are listed in Table 2, [10].

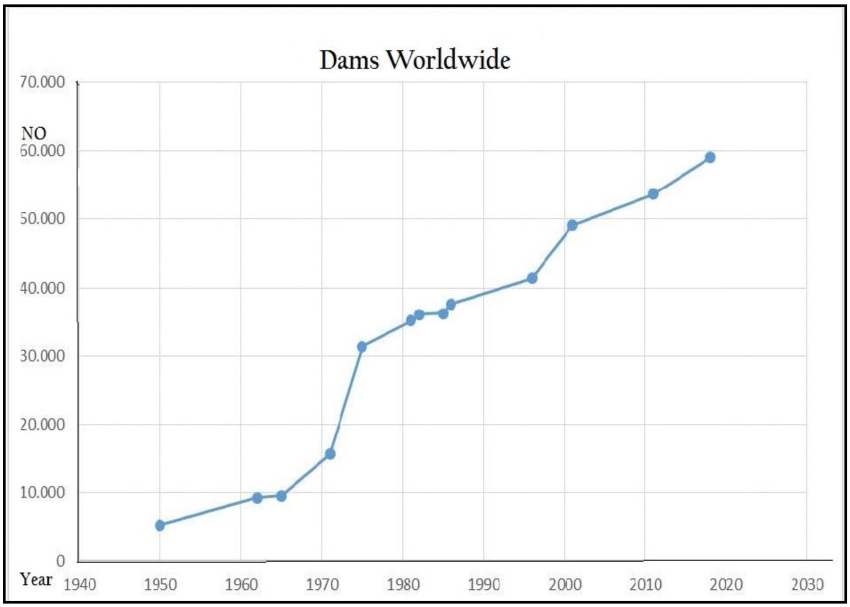

Figure 4: Growth of Numbers of Dams in the world (1950-2020) [9].

Table 1: Number of large dams in the world showing their purpose [10].

Code	Desecription	Number of Dams with Sole Purpose	Number of Multiple-Purpose Dams
C	Flood Control	2524	4778
F	Fish farming	41	1395
H	Hydropower	5786	3932
I	Irrigation	14562	5954
N	Navigation	97	580
R	Recreation	1350	2942
S	Water Supply	3285	4330
T	Tailing	63	8
X	Others	1540	1214

Table 2: The twenty highest dams in the world, their purpose and country [10].

Dam name	Height (m)	Purpose	Country
Rogun	335	HI	Tadjikistan
Bakhtiyari	315	HC	Iran
Jinping 1	305	HC	China
Nurek	300	IH	Tadjikistan
Lianghekou	295	H	China
Xiaowan	294	HCIN	China
Xiluodu	286	HCN	China
Grande dixence	285	H	Switzerland
Baihetan	277	H	China
Inguri	272	HI	Georgia
Diamer-bhasha)	272	HIS	Pakistan
Yusufeli	270	H	Turkey
Manuel Moreno Torres (Chicoasén)	262	H	Mexico
Nuozhadu	262	HCN	China
Tehri (thdc)	260	IH	India
Hacixia	254	H	China
Mauvoisin	250	H	Switzerland
Laxiwa	250	H	China
Deriner	249	H	Turkey
Mica	243	H	Canada

4. Dam Safety and Dam Hazards

The great number of dams which have been built already in the world today or those to be built in the future have been subject of special emphasis on the questions of dams' safety and their hazards. The terms "Safety" and "Hazards" are interrelated; the first means the quality of averting or not causing injury, danger, or loss; the second may be said to denote something unavoidable, danger or risk. The question of dam safety concerns the dam only, while the term dam hazard is much wider as it encloses in addition to losing the dam itself the risk of flooding of the downstream valley causing life loss, material and property losses and environmental damage. In this case, the reservoir behind the dam is called into play as an essential element in deciding the magnitude of the hazard.

Dam safety is normally controlled by the owner through proper selection of the site, ensuring good design, proper construction followed by competent management, applying appropriate inspections and repair measures; all these actions are to be done following recognized specifications, codes, laws, and regulations. The dam hazard question however, must be followed by national authorities and governments whose responsibility does not stop at issuing the required laws and legislations but also ensuring the correct following of them by the dams' owners, and this extends to give serious thoughts to the possibility of failure of the dam, even if the probability of such an event is infinitesimal. Planning for such a remote possibility includes the performance of dam break studies as a prerequisite to any work concerning flood zoning, early flood warning, evacuation plans, and rescue and salvage measures to reduce the damages to the minimum possible level. Emergency Action Plans (EAP) to meet all these must always be available, updated, and ready, and a certain level of preparedness shall be kept always to meet emergencies. The failure of a dam and the hazards it involves could take the dimensions of a national catastrophe in the absence of such measures.

In summary, the objectives of all this, as put by ICOLD, are the following:

i. To control the release of damaging discharges downstream of the dam,
ii. To restrict the likelihood of events that might lead to a loss of control over the stored volume and the spillway and other discharges,
iii. To mitigate through on-site accident management and/or emergency planning the consequences of such events if they were to occur [11].

These fundamental objectives and principals of dams' safety and mitigation of their safety hazards should apply to all dams and their associated reservoirs throughout their lifetime, whether during planning, design, construction phase, or their commissioning and operation, and having always in focus minimizing any loss or damage downstream in the event of uncontrolled accident or failure.

5. Dam Failure

Dam failure is the sudden and rapid uncontrolled release of water impounded by the dam due to collapse of the dam itself or, in some cases due to an upstream land slide into the reservoir with or without the collapse of the dam. Such an uncontrolled release of water may result from insufficient or faulty design, inferior construction procedures, malfunctioning of one or more components of the dam, or as in the case of landslides, insufficient investigation of the stability of the reservoirs`' slopes. Insufficient design in its turn may be caused by one of the following:

i. Misunderstanding of the geological conditions of the foundation of the dam. This leads to wrong or deficient foundation treatment, and seepage controls arrangements, or unexpected deformation of these foundations.
ii. Erroneous calculation of the inflow designed flood. This can result in overtopping of the dam due to insufficient spillway capacity and/ or

insufficient free board.

iii. Misinterpretation of the seismic conditions of the area and adopting seismic criteria which are not compatible with such seismic conditions.

Faulty design may include among many other things the use of improper factors of safety and method of stability analysis, selection of inappropriate construction materials, insufficient drainage arrangements of the foundation and the dam body. Similarly, inferior quality control during construction is an obvious cause which can lead to failure of a dam and needs no further comment.

Mismanagement in the operation of a dam can lead also to complications as far as safety is concerned. This may reflect on daily operation of the various components of the dam, or during flood events or maybe after a seismic event. Ensuring good management requires the service of experienced and seasoned personnel that are not only familiar with such routines but who will also carry out all required inspections and detect any anomaly in the behavior of the dam at an early stage and apply the proper remedies or report the problem to the higher technical level if it is beyond their available means.

Table 3 highlights some of the registered major dam failures in the world showing year of occurrence, the country, and number of fatalities during (1950-2019).

Table 3: List of major dam failures in the world (1950-2018), excluding Tailing dams` failures and those failures with fatalities less or equal 10, [12].

Dam	Date Fa Date	Location	Fatalities	Details
Machuchu-2 Dam	1979	Morbi, India	5000	Heavy rain and flooding beyond spillway capacity
Val de Stava Dam	1985	Tesero, Italy	286	Poor maintenance and low margin for error in design. Outlet pipes failed leading to pressure on dam
Kantale Dam	1986	Kantale, Sri Lanka	180	Poor maintenance, leakage, and consequent failure destroyed over 1600 houses and 2000 acres of paddy fields
Zeyzoun Dam	2002	Zeyzoun, Syria	22	2000 individuals displaced and over 10,000 directly affected
Shakidor Dam	2005	Pasni, Pakistan	70	Sudden and extreme flooding caused by abnormally severe rain
Gusau Dam	2006	Gusau, Nigeria	40	Heavy flooding. Approximately 500 homes were destroyed, displacing 1,000 people
Situ Gintung Dam	2009	Tangerang, Indonesia	98	Poor maintenance and heavy monsoon rain
Kyzyl-Agash Dam	2010	Qyzylaghash, Kazakhstan	43	Heavy rain and snow melt. 300 people were injured and over 1000 evacuated from the village
Patel Dam	2018	Solai, Kenya	47	Failed after several days of heavy rain
Xe-Pian Xe-Namnoy Dam	2018	Attapeu Province, Laos	36	Saddle dam under construction collapsed during rainstorm. 6600 people homeless, 98 missing
Note: For a more comprehensive list of dam failures, reference can be made to [12].				

Some major failures were excluded from this table as the number of fatalities was less or equal to ten, although they had caused extensive material losses. The case of Teton dam failure mentioned above is worth considering. The low number of fatalities of only 11 was due to early warning and efficient evacuation of the people who were at risk. The dam was located on Snake river, Idaho, USA, and it was an earthfill dam 100 meters high completed in 1976 and collapsed during first filling. Its failure caused the release of 296,000m^3 of water; fortunately, 30,000 people were evacuated from the downstream flooded area, and the number of human losses was only eleven. The failure was due to insufficient protection against foundation seepage and the misinterpretation of the characteristics and properties of the filling material. According to the data of the National Performance of Dams Program (USA), the number of failures and height of the failed dams from 1848 until 2017 in the United States are as shown in Table 4.

Table 4: NPDP Dam Failure Database summary (USA) [13].

Data Summary	Value
Period of Record	1848-2017
Number of Dam Failures All Dams < 15 m Dams > 15 m	 1645 1546 99
Number of States Having Dam Failures	50
State with the Most Failures	Georgia
States with the Fewest Failures	Hawaii Louisiana
Earliest Known Dam Failures	1848

Further data from the same source give more details on Dam Failure-Fatality statistics in USA as indicated in the following Table 5 for the period 1850-2016.

Table 5: Summary of the Dam Failure-Fatality Dataset (USA) 1850-2016 [14].

Parameter/ Summary	Value(s)
Period of Record	1850-2016
Number of years of Record	167
Number of Dam Failure Resulting in Fatalities	63
Frequency of Occurrences per year of Dam Failure Events	38%
Range of the Number of Fatalities that occurred over the Period of Record	1-2,209
Total Number of Fatalities that occurred over the Period of Record (A range is shown based on the variation of the estimates of the number of fatalities that occurred)	3,432-3,736
Average number of Fatalities per Year over the Period of Record	20.6-22.4

It is evident from these data and field investigations that the number of fatalities that may result from dam failure is function of number of factors; these include (but are not limited to):

i. Size of the dam and reservoir.
ii. Severity (depth, velocity, and arrival time) of flooding that occurs downstream.
iii. Population at risk at the time of the failure (time of year, time of day).
iv. Location of the population at risk downstream of the dam and in the area that is inundated.

v. Location of safe havens.
vi. Effectiveness of local emergency management services.

6. Dam Incidents

Dam incidents, contrary to dam failures, are events, which occur during the lifetime of a dam which threatens its safety, but they are manageable at an early stage, and the dam may be saved. So, dam incidents have considerable engineering and safety value in that they provide insight into the structural and functional integrity of dam systems and their operation.

Such events may be experienced in case of an earthquake event resulting in large settlement or the appearance of distress signs on the dam body and the appurtenant structures and/or the mechanical and electrical equipment, similarly these can happen during very high floods causing damage to the water ways or jamming of one or more gates, or even the uncontrolled release of water causing damage to the downstream.

Incidents may happen even during normal operation such as landslides into the reservoir without causing overtopping, ether because the volume of the slide is not so large or the reservoir level being low during the event, in which cases these mark near misses.

More incidents may happen due to errors on the part of the operators by not following the prescribed rule curves and the operation instructions. In this, category falls the early filling of the reservoir not anticipating further flooding and the very fast rate of drawdown which could affect dam stability; and even in some cases affecting the reservoir slopes stability due to saturation of the soil mass increasing its weight while stabilizing forces i.e. balancing water pressure and resisting friction force are reduced suddenly. Negligence and bad maintenance are another human failing, which may lead to the deterioration of the dam’s materials, equipment and essential safety devices such as drainage systems.

Documenting and reporting all incidents even the minor of them is essential to ensure dam safety; and keeping log books of all observations and events are most important allowing proper diagnosis in similar future cases. Immediate actions by the operators must be undertaken to execute the necessary remedies or report the issues that are beyond their capacity to the higher technical levels. Besides repairs, modifications of the design may have to be done in serious cases. An example to such case is modifications made to increase the spillway capacity or raise the dam itself. Stabilizing landslides or rockfalls that shows signs of danger may have to be treated to increase their stability; an example to such a case is Derbendikhan Dam landslide in Iraq.

Drilling new drainage holes to replace clogged once, or performing remedial grouting works may also prove necessary during operation, and it takes diligent operators to first diagnose the need for such works through careful follow up of the instrumentations readings and their intelligent interpretation.

Keeping records of the frequency and impacts of events related to the general performance of dam is of prime value. Of such events; the occurrence of very high waves generated by violent winds, impacts of such winds on the operating systems, ice sheets jamming waterways, trash, and debris, which clog trash racks or intakes. Such reports help in finding and implementing technical solutions so that the dam safety as a whole is not compromised in similar situations.

Importance of the human factor and the value of training in enhancing "Dam Safety" play a very important role in avoiding dam incidents, which can lead to inadequate risk management. This is highlighted by the following paragraph quoted from a recent article on the importance of the human factor:

"Inadequacies" in risk management may be classified into three types:

1. Ignorance involves being insufficiently aware of risks. This may be due to aspects of human fallibility and limitations such as lack of information, inaccurate information, lack of knowledge and expertise, and unreliable intuition. Complexity can also contribute to ignorance.
2. Complacency involves being sufficiently aware of risks but being overly risk tolerant. This may be due to aspects of human fallibility and limitations such as fatigue, emotions, indifference, and optimism bias ("it won't happen to me"). Pressure from non-safety goals can also contribute to complacency.
3. Overconfidence involves being sufficiently aware of risks, but overestimating the ability to deal with them. This may be due to aspects of human fallibility and limitations such as inherent overconfidence bias, which results in overestimating knowledge, capabilities, and performance"

The article goes on to state:

"Counterbalancing" the drivers of failure noted above, the human factors contributing to system capacity for safety generally emanate from what is routinely referred to as "safety culture". The general idea of safety culture is that individuals at all levels of an organization place high value on safety, which leads to a humble and vigilant attitude with respect to preventing failure. For such a safety culture to be developed and maintained in an organization, the senior leadership of the organization must visibly give priority to safety, including allocating the resources and accepting the tradeoffs needed to achieve safety", [15].

While all of this is well said and true; it is of paramount importance to realize that training in safety awareness and procedures is one of the most important factors in creating this "safety culture", and it must be realized that such training is required at all organizational levels, not only on the level of operators of the dam, but the whole hierarchy up to decision makers at the top.

7. The question of loss of life and its evaluation

Loss of life due to dam failures has been experienced for a long time over history. Although, Table 1 shows some of the important registered cases of failure with their corresponding number of fatalities since the middle of the last century, other registered data indicates considerable life losses even before that [12], not to mention those cases which were left unregistered. The question of life loss did not take the full attention of societies before for many reasons. First, not such a great number of dams were built at the time, which meant also that not so many possibilities of failures existed. Second, many failures went unnoticed by the public due to poor communications or limited public news media; Third, the exponential growth in dam building activity since the beginning of the last century.

In spite of the advances in knowledge and construction technology, frequent failures and accidents did happen which were associated with loss of life, and the wide reporting of human fatalities drew much more attention to the question of dams as threats to human societies.

The worth of building new dams came to be examined not only by investors and banks but also by environmentalists, welfare societies, health organizations and insurance companies, and the question of loss of life became central in the evaluation of dam risks. Governments and owners in addition to other stakeholders started to take a keen interest in this question which led to the formulation of new methods of assessing dams' safety and their risks especially oriented towards life loss.

Of the many governmental bodies and dam owners who have taken a keen interest in this question were the United States Bureau of Reclamation (USBR) and the United State Army Corps of Engineers (USACE). Their approach involved quantifying the failure likelihood category level of any dam as a function of both the critical loading conditions leading towards failure and the chance of its occurrence. A chance of occurrence probability of 1:10,000 as the threshold separating moderate likelihood from high likelihood zones of failure was accepted by both entities as a reasonable criterion for acceptable loss of life [16]. The concept of acceptable risks to human life has been the subject of research conducted by the Health and Safety Executives, UK (HSE) since the 1970's. Although their research was mainly oriented towards risks in the industrial and nuclear sectors it had contributed to the definition of tolerable risk and acceptable risk in general in other areas such as dams. In this respect, HSE reckons that the individual risk/annualized failure probability guideline is generally taken as 1 in 10,000 per year. In the water resources industry, this threshold seems to describe an agreeable guideline. Further work was done to relate this concept to dams and their risk assessment by the Construction and Industry Research and Information Association, UK (CIRIA). Their work and findings were issued in the UK as a report in 2000 "CIRIA 568, Risks and Reservoirs".

To quantify the risk paused by any dam to life and property, a matrix has been suggested by the USBR and USACE, shown in Figure 5. The location of dam under consideration on this matrix is obtained after thorough examination of the dam conditions so that the likelihood of failure is estimated with the help of indicators outlined in Table 6, and similarly the category or level of consequences is be obtained by referring to Table 7 where estimating of these consequences is based on loss of life taken as a measure of the risk posed by the dam [17].

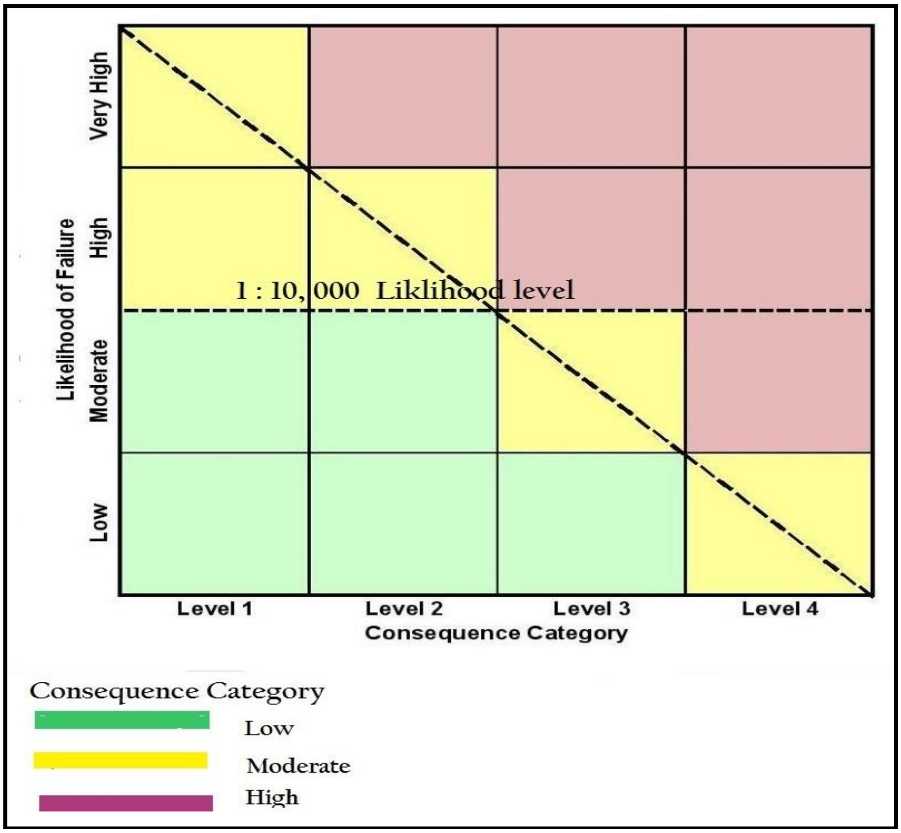

Figure 5: Quantitative Risk Matrix Format.

Table 6: Failure likelihood Categories.

Category Name	Description
Low Likelihood	The possibility cannot be ruled out, but there is no compelling evidence to suggest it has occurred or that a condition or flaw exists that could lead to its development (e.g., a flood or an earthquake with annual exceedance probability more remote than 1.0 x 10^{-5}/yr would cause failure)
Moderate Likelihood	The fundamental conditions or defect is known to exist; indirect evidence suggests it is plausible; and key evidence is weighted more heavily towards unlikely than likely (e.g., a flood or an earthquake with annual exceedance probability between 1.0 x 10^{-5}/yr and 1.0 x 10^{-4}/yr would likely cause failure)
High Likelihood	The fundamental conditions or defect is known to exist; indirect evidence suggests it is plausible; and key evidence is weighted more heavily towards likely than unlikely (e.g., a flood or an earthquake with annual exceedance probability between 1.0 x 10^{-4}/yr and 1.0 x 10^{-3}/yr would likely cause failure)
Very High Likelihood	There is direct evidence or substantial indirect evidence to suggest it has occurred and/ or likely to occur (e.g., a flood or an earthquake with an annual exceedance probability more frequent (greater) than 10 x 10^{-3}/yr would likely to occur.

Table 7: Consequences Categories (numbers refer to casualties).

Category Name	Category Description
0	No significant impacts to the downstream population other than temporary minor flooding of roads or land adjacent to the river
1	Although life threatening flows are released and people are at risk, loss of life is unlikely
2	Some life loss is expected (in the range of 1 to 10)
3	Large life loss is expected (in the range of 10 to 100)
4	Extensive life loss is expected (greater than 100)

8. Conclusion

History of dams' construction is full of mixed cases of successes and failures, and the chances of more failures still exist even today. The great number of dams that have been built during the last century and the ones being constructed in the two decades of the twenty century have resulted in a tremendous increase in the number of these dams all over the world. Such increase was dictated by the increasing demand for producing food to the growing populations, and their need for electric power. The need for flood protection is also demanded by the growing populations inhabiting river valleys. The risks posed by these dams to people, and material properties have grown in lieu of the large number of these dams. Even with the vast improvements in the ways of collection of data used, the much improved methods of design and the more refined methods of analysis, the probability of failure of dams, even if it is reduced, remains as a threat to human societies. This leads to concentrate attention on "Dam Safety" as a way of reducing dangers to human lives and material properties.

Dam Safety issues, as we visualize them, are not only taking care of the proper design and good construction of dams only, but extending this to their safe management and operation, in addition to more understandings of the natural hazards impacting them such as floods and earthquakes. The increasing number of aging dams adds another worry to dam owners due to the reduced competence of these dams as result of passage of time and deterioration of their materials and ancillaries, which merits special attention.

There exist now tremendous volume of records documenting case histories of dam incidents and failures. Lessons learned from these case histories have led to improved guidance and technical understanding, and ultimately to safer dams.

ICOLD (1995) listing shows that dam failures had continued over the previous four decades with less than 0.5% of the 12,138 large dams built between 1951 and 1986 [18]. However, this is still 59 failures around the world. Judging by the magnitude of losses resulting from these failure it is a clear that such statistics are misleading unless they are qualified with statistics of human fatalities and property damages associated with them which in these cases were. tremendous.

Long-term concerns over improving dam safety include, among other things, taking seriously future "Climate Change Impacts" on dams and the decommissioning of aged dams.

More emphasis over the need for safety measures and diligence in the future developments of dams need no further explanation. With the growing need for water for the expanding industry and agriculture to satisfy, the increased population dams will continue to be built and raised in height exasperating the need for stringent "Safety Measures".

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Dam Safety and Dams Hazards

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Abstract

Dam safety hazards towards human communities have increased tremendously during the last decades. They have resulted from dam safety problems leading to failure and being exasperated by the large losses at downstream areas due to increasing populations and land use. In this work, an attempt is being made to review the procedures being followed to reduces these hazards by improving dams safety standards. Classification of dams by their potential hazards are explained as used today for prioritize remedial actions in various countries of the world. The guiding principles of these classification are indicated and they are based on height of such dams and their storage and linked to the potential damage and harm they can create. Normally such classification and follow up actions are supported by various legislations and regulations issued by the respective governments. Moreover, conventions signed by riparian countries promote cooperation on mitigating safety problems of dams on transboundary rivers. Examples of such legislations and conventions are mentioned. Looking for having safer dams is an objective continually which is being pursued as more dams are needed in the future while existing dams continue to serve their objectives. Therefore, using lessons learned from previous failures is recommended taking the question of loss of life as a main doctrine.

Keywords: Dam Safety, Dam Hazards, Dams Classification, Potential Hazard, Loss of life.

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1. Introduction

Dams have served mankind since ancient times. Irrigation of agricultural land was the main drive for building dams in those days, and it remains as a very important tool for such use until today. By damming a river and controlling its flows this will harness flood waters of the wet seasons to utilize it in the dry periods. Flood control function of dams has saved humanity from great losses of lives and spared rich lands from inundation and damage. The importance of dams grew in the last century due to the need for electric power needed in industry and human applications such that a large percentage of the world electric power is generated by dams nowadays. According to dam's statistics of the International River Organization, there were in 2007 the following numbers of dams:

Global number of dams: c. 800,000
Global number of large dams: >40,000
Global number of major dams: >300

The same reference states that there was in 2007 more than 50,000 dams higher than 15m or more, and that some of them dated back to centuries, but most were built after World War II. About 5,000 dams have heights of 60m or more; another 350 of such giants were under construction in 2014 [1], [2].

Although dams are a blessing to the countries rich with rivers, they create hazards to population living in their downstream flood plains. The subject of dam safety, therefore, must be approached with caution and prudence. Dam safety cannot be concerned only with the structure itself, but it should be considered in relation to the population at risk in the downstream flood plain.

2. Dams Classification according to their size and their potential hazards

Different authorities have different classification systems for dams with respect to safety hazard`s evaluation related to them. These classifications serve either design, or licensing purposes, or may be used for other goals such as prioritizing remedial actions and budgeting planning. Most of these systems adopt criteria based on dam's parameters such as height or size of reservoirs and/or the potential hazards causing loss of life and economic losses. Few of these systems are presented here to introduce the philosophy behind dams safety procedures currently applied in the world today.

2.1 Classification of the International Congress on Large Dams (ICOLD)

The International Commission on Large Dams (ICOLD), considers any dam is large dam if its height is 15 meters or more measured from the lowest point of foundation to top of the dam. Dams with a height of 10 m or more are considered as high dams, if they satisfy one of the following criteria or a combination of more than one of them as set out in Table 1.

Table 1: ICOLD large dams classification criteria for dams >10 m in height [3].

Criterion	Requirement
Length of crest	> 500 m
Volume of reservoir	> 1 million m^3
Design flood	> 2000 m^3/sec.
Geological conditions	Difficult
Other considerations	Unusual Design

In considering the question of potential hazards posed by dams, ICOLD has modified these basic criteria to link the size of a dam to the hazards it could create and has adopted modified criteria based on the French Committee on Dams and Reservoirs' guidelines [3]. These guidelines consider the two parameters (H), the maximum height of the dam in meters, and (V) which is the reservoir's volume in m^3 in defining a numerical index $H^2\sqrt{V}$ to rate the Potential Hazard Classification (PHC). The relationship may be obtained by using semi-log paper; (V) on the X-axis and (H) on the Y-axis as shown in the chart given in Figure 1, this chart may be read in conjunction with Table 2 for obtaining the Potential Hazard Classification (PHC) for small dams not exceeding 25m in height and having reservoir volume not more than 100 million m^3.

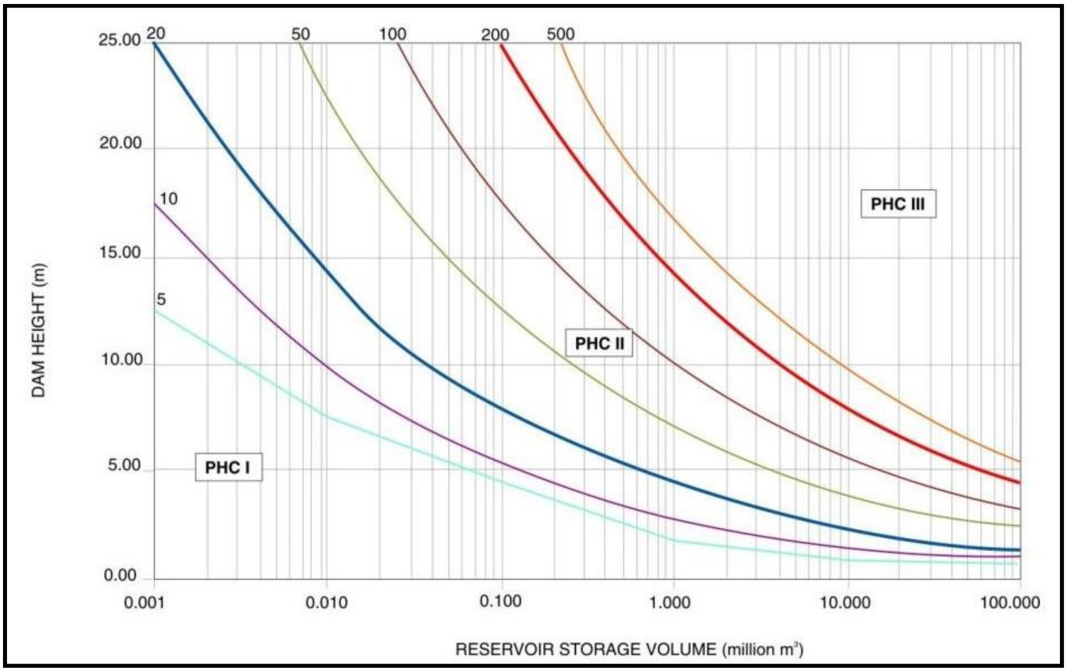

Figure 1: Relationship $H^2\sqrt{V}$ for small dams [3].

Table 2: Potential Hazard Classifications (PHC) [3].

Component	**Potential Hazard Classification (PHC)**		
	Low- (I)	Medium- (2)	High- (III)
$H^2 \sqrt{V}$	$H^2\sqrt{V} < 20$	$20 < H^2\sqrt{V} < 200$	$H^2\sqrt{V} \geq 200$
Life Safety Risk (number of lives)	~ 0	< 10	≥ 10
Economic Risk	Low	Moderate	High or extreme
Environmental Risk	Low or Moderate	High	Extreme
Social Disruption	Low (rural area)	Regional	National

This guideline; however, modifies ICOLD's definition for small dams given in Table 1, and qualifies them to be high dams if they follow the criterion shown below: Dams with height $5 < H < 15$ m and $V > 3hm^3$.

Accordingly, the differentiation of small dams from large dams follows the boundaries in Figure 2. This classification is related directly to hazards posed by these dams and therefore, it is a better classification system than the original ICOLD classification.

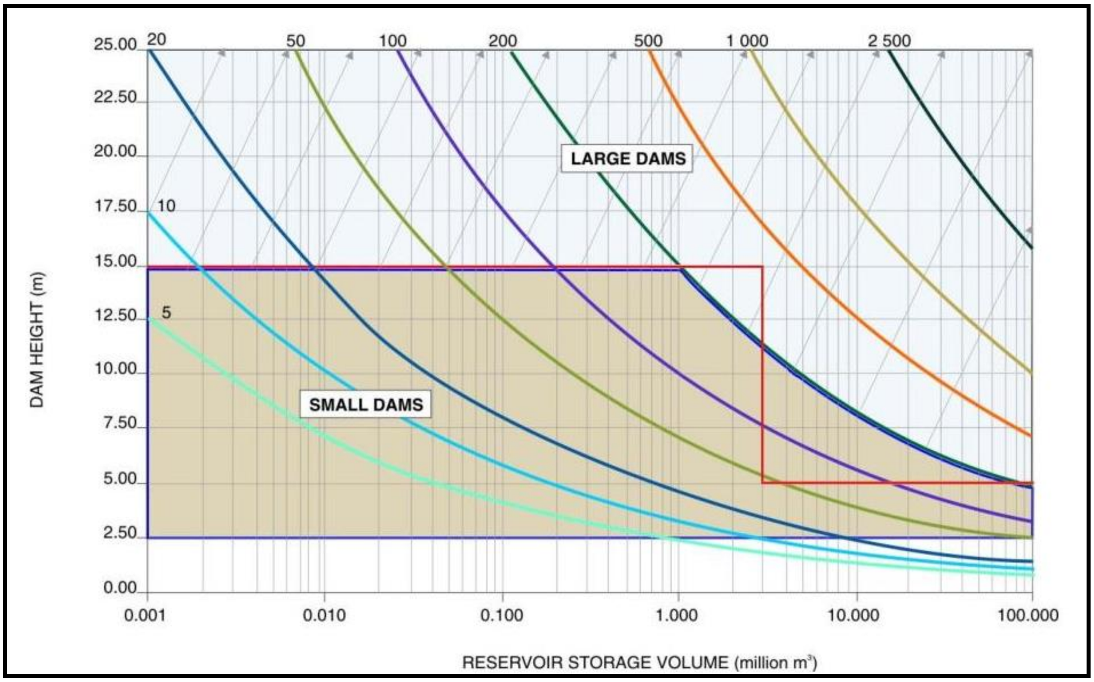

Figure 2: Classification of small and large dams [3].

2.2 Classification of the Federal Emergency Management Agency (FEMA) [4].

This agency was established in the USA in April 1979 with the objective of protecting people from hazards associated with nuclear power plants, the transportation of hazardous substances in addition to protection from and mitigating hazards associated with natural disasters. Enhancing public safety that is caused by dams became also one of its main concerns.

The classification of dams adopted and used by FEMA follows two criteria:

- Loss of life
- Economic, Environmental and Lifeline losses

Three classification levels are used, which are: LOW, SIGNIFICANT, and HIGH, listed in order of increasing adverse incremental consequences. The classification levels build on each other, i.e., the higher order classification levels add to the list of consequences for the lower classification levels, as noted in Table 3, and described below.

Table 3: Classification of dams according their hazard potential [4].

Classification	**Loss of Human Life**	**Economic, Environmental, Lifeline losses**
A- Low	None Expected	Low and Generally limited to Owner
B- Significant	None Expected	Yes
C- High	Probable, one or more expected	Yes

A. Low Hazard Potential

Dams assigned to the low hazard potential classifications are those where failure or miss- operation results in no probable loss of human life and low economic and/or environmental losses; but losses are principally limited to owners’ properties only.

B. Significant Hazard Potential

Dams assigned to the significant hazard potential classifications are those dams where failure or mis-operation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classified dams are often located in predominantly rural or agricultural areas but can be located in areas with population and significant infrastructure.

C. High Hazard Potential

Dams assigned to the high hazard potential classification are those where failure or mis-operation will probably cause loss of human life in addition to economic, environmental and lifeline losses.

This Hazard Potential Classification System recognizes that the failure or misoperation of any dam or water-retaining structure, no matter how small, represents a potential danger to downstream life and property. Whenever there is an uncontrolled release of stored water, there is always the possibility, regardless of how unexpected, of someone being in the path of the discharge. The difference between the Significant and High Hazard Potential Classification levels is the loss of life criterion; whereby a high hazard dam may cause probable loss of human life and in a significant hazard potential dam such loss is not expected.

2.3 Other Classification Systems

Many States in the United States have their own systems of classification for dams, but all these dams fall under the jurisdiction of the Federal Emergency Management Agency (FEMA). These systems are mainly intended for licensing the construction of new dams, regulating the construction procedures, and setting rules for the operation and maintenance of such a dam. All have in view public safety and the reduction of the hazards posed by these dams to the public [5], [6]. Countries such as UK, Australia, Canada, and most of the European countries also have their classification with the same objectives. In Table 4, a summary of guidelines adopted by the Australian National Committee on Large Dams (ANCOLD for the classification of the potential hazards of dams is given. These guidelines introduce the term Population at Risk (PAR) as the criterion on which hazard classification is built. Population at Risk means all the population in the downstream flood plain, which is subjected to dam failure hazards [7], [8].

Table 4: Dam Hazard Categories according ANCOLD [7], [8].

Population at Risk (PAR)	**Severity of Damage and Loss**			
	Negligible	**Minor**	**Medium**	**Major**
0	Very Low	Very Low	Low	Significant
1-10	Low Notes 1 and 4	Low Notes 4 and 5	Significant Note 5	High C Note 6
11-100	Note 1	Significant Note 2 and 5	High C Note 6	High B Note 6
101- 1000		Note 2	High A Note 6	High A Note 6
>1000			Note 3	Extreme Note 6

Note 1: With a PAR of 5 or more people, it is unlikely that the severity of damage and loss will be: Negligible.

Note 2: "Minor" damage and loss would be unlikely when the PAR exceed 10.

Note 3: "Medium" damage and loss would be unlikely when the PAR exceeds 1000.

Note 4: “Change to *significant* where there is the potential for one life being lost is recognized.

Note 5: Change to High where is the potential for one or more lives being lost.

Note 6: Refer to section 2.7 and 1.6 of ANCOLD Guidelines on the consequences of Dam Failure for an explanation of the range of High Hazard Categories.

In the previous table the likelihood of occurrence of hazardous incidents causing medium or major damage and loss is classified into categories of:

i. High A; Almost certain. The event is expected to occur in most circumstances,
ii. High B; Likely. The event will probably occur in most circumstances and,
iii. High C; Possible. The event could possibly occur at some time

It follows from all this that Public Safety is looked at in almost all countries in most careful and scrutinizing eyes. Concerning dams, they should be classified according to their potential impacts on the communities downstream as explained already. Such classification must be adopted and put into application; for new dams, this is done by thorough investigations and studies, safe designs, and proper construction procedures, which are commensurate with the class of the dam. For existing dams safe operation and routine safety checking and maintenance must be conducted as required by their operation and maintenance manuals and even by introducing new innovations in these fields such as new types of monitoring systems i.e. new instruments and/ or remote sensing methods. In the case of any anomaly is being discovered then all required corrective actions should be performed promptly to upgrade the safety conditions of the dam under consideration.

Thousands of dams have been built during the last 100 years and some of them are suffering from clear signs of aging and becoming serious threats to the Public. If corrective measures seem to be impractical or very costly, then the question of decommissioning them should be addressed seriously.

3. Dam safety legislations and Dam Safety Program

In great number of countries dams safety classification and required measures of public safety protection are defined by legislations in the form of laws, regulations, and guidelines. The classification systems mentioned in the previous section or very similar ones are embodied in such legislations in many countries such as USA, UK, France, Australia, and the latest bill on dam safety in India (2019) [9].

In many other countries such as Turkey, no specific legislation for dam classification and their required safety measures are available but the general requirements are identified following many laws and regulations, such as Protection against Flooding Law (1943), Civil Defence Act (1958), Measures and Assistance Regarding Natural Disasters affecting General Public Life Precautions Act (1959), DSI Regulation on Protection against Flooding (1982), The Environmental Law (1983), and Regulation on the Environmental Impact Assessment (2003) [10].

Other countries, Iraq for example, unfortunately have no classification system of their own on which dam safety legislation are based in spite of the numerous dams they own and the fact that they threaten the safety of millions of people. It is therefore time to sound the alarm so that the respective governments of these countries pay heed to this important question.

In Riparian countries sharing large transboundary river the classification and safety legislations of dams on transboundary rivers in the form of conventions and mutual cooperation take special importance as any safety threat in one of these countries will have its implications on the other sharing countries. Many such conventions have been developed and signed for river basins in the world. One example is the “Danube River protection Convention (1998) between fifteen European countries sharing the river basin which stipulates among other important issues preventive measures to control hazards originating from accidents involving floods, ice, or hazardous substances [11], [12].

In the Eastern Nile sub-basin spanning Egypt, Ethiopia, Sudan, and South Sudan, thirty transboundary dams operate in the region, including, Roseries in Sudan, Aswan in Egypt, and many smaller dams no higher than 15 meters, with a combined storage capacity of 210 billion cubic meters. More, including on the mighty Blue Nile, are being built such as the Renaissance in Ethiopia. Attention to dam safety is critical issue which concerns the design or the inadequate monitoring and maintenance which can increase the risk of dam failure and have significant flood consequences and affects river bank settlements, fisheries, power generation, agriculture, the environment, and the overall regional economy.

Through its dam safety program, the Nile Basin Initiative carried out by the Eastern Nile Technical Regional Office (ENTRO) with support from World Bank’s Cooperation in International Waters in Africa program and CIWA works to build technical capacity and establish national and regional safety norms through national dam safety units so these countries can standardize dam safety management, safeguarding against such threats as dam breaches that put at risk a Nile Basin population that may double in 20-30 years.

CIWA is the Cooperation in International Waters in Africa program which is a multi-donor trust fund representing a partnership between the World Bank and the governments of Denmark, the European Union, Norway, Sweden, the Netherlands, and the United Kingdom.

Additionally, the collaboration among countries in the dam safety program allowed technicians across the region to discuss technical operational issues beyond dam safety, further increasing trust among the participating countries [13].

4. Dams Hazards and the Question of Loss of Life

Hazards caused by dams are now quantified by the loss of life that could result from their failure. Classification of dams according their potential hazards was treated in a previous paragraphs. But these hazards, as it can be seen, are not dependent only on the dam structure itself, but they are also linked to the nature of the downstream flood plain; its topography and its extent. Other important factors in this respect are the level of occupancy of the flood plain by people and how many of them are directly exposed to the danger of drowning. These hazards can be also modified by the administrative procedures and regulations which have been enacted to deal with such emergency, and the degree of preparedness for dealing with natural disaster events particularly related to flooding due to dam failures. Therefore it may be said that loss of life is influenced by the following factors:

i. The number of people occupying the dam failure flood plain,
ii. The severity of the flooding,
iii. The time available to warn people to the incoming flooding, and,
iv. The degree of preparedness to deal with such an emergency.

The procedure for estimating loss of life due to dam failure has been developed to great extent in the United States and it is based upon data from U.S dam failures. This procedure is composed of seven steps which are:

i. Determination of dam failure scenarios to evaluate.
ii. Deciding on time categories for which losses of life estimates are needed.
iii. Determination when dam failure warnings would be initiated.
iv. Finding out the area flooded for each dam failure scenario.
v. Estimating number of people at risk for each dam failure scenario and time category.
vi. Estimating the number of fatalities by applying empirically based equations or methods.
vii. Evaluating uncertainty.

According to Graham [14], each of the above steps is explained, which are as follows:

Step 1: Determination of dam failure scenarios to evaluate;

Loss of life may need two scenarios; failure of the dam with full reservoir during normal weather conditions and failure during large flood that overtops the dam.

Step 2: Deciding on time categories for which loss of life estimates are needed;

Normally, the number of people at risk in the downstream changes according to the season, and to the week and the day. So, the study must be repeated for selected time categories according to the varying occupancy of the flood plain. In the time category studies (Day) and (Night) estimates shall be made. The degree of occupancy can vary considerably between (Day) and (Night) due to the number of computers who are present at their works during the day but leave the area to outside locations for the night which changes the degree of occupancy. The initial time of sounding the alarm must be repeated in such studies since people are more likely to respond to this alarm during daytime than during night.

Step 3: Determine when dam failure warnings would be initiated;

The most important factor affecting loss of life is when the warning of the incipient failure has been initiated. Based on data collected from dam failures in the U.S since 1960 and data from Vajont Dam failure in Italy, Malpasset Dam failure in France and St. Francis Dam failure in California it is found that alarm sounding is less likely during the night than during the day, that timely sounding of the alarm for dams, with smaller reservoirs with little flood storage and smaller catchment basins, are also less likely. Limited data also shows that timely warning is less likely for concrete dams than for earth dams. Table 5 shows guidelines, which help in estimating warning time to be used in loss of life studies resulting from dam failures. The data are for earth fill dams and based on actual collected data.

Table 5: Guidelines for estimating warning time in loss of life studies for dam failure scenarios (Earth fill Dams).

Mode of Failure	**Special Considerations**	**Time of Failure**	**When Would Dam Failure warning is initiated**	
			Many Observers at dam site	**No Observers at dam site**
Overtopping	Drainage area < 260 km^2	Day	0.25 hrs. before dam failure	0.25 hrs. after fw* reaches populated area
	Drainage area < 260 km^2	Night	0.25 hrs. before dam failure	1 hrs. after fw reaches populated area
	Drainage area > 260 km^2	Day	2 hrs. before dam failure	1 hrs. before dam failure
	Drainage are> 260 km^2	Night	1- 2 hr. before dam failure	0- 1 hr. before dam failure
Piping (full reservoir, normal weather)		Day	1 hr. before dam failure	0.25 hrs. after fw reaches populated area
		Night	0.5 hr. after dam failure	1.0 hr. after fw reaches populated area
Seismic	Immediate Failure	Day	0.25 hr. after dam failure	0.25 hr. after fw reaches populated area
		Night	0.5 hr. after dam failure	1.0 hr. after fw reaches populated area
	Delayed Failure	Day	2 hrs. after dam failure	0.5 hr. before fw reaches populated area
		Night	2hr. after dam failure	0.5 hr. before fw reaches populated area

Notes: "Many observers at Dam" means that tender lives on high ground within site or the dam is visible from the homes of many people or the dam crest serves as a used roadway. These dams are typically in urban areas. "No Observer at Dam" means there is no dam tender; the dam is out of site. These dams are at remote areas.

The Abbreviation * "fw" stands for flood water.

Step 4: Finding out the area flooded for each dam failure scenario.
Inundation map or other description of the flooded area is needed for each scenario. For such a requirement existing dam break studies and maps are useful.
Step 5: Estimating of number of people at risk (PAR) for each failure Scenario and time category. PAR is defined as the number of people occupying the dam failure flood plain prior to the issuance of any warning. Normally, this number varies during a 24 hours period. This is most likely dependent on the time of the year, day or week and time of the day during which the failure occurs. Sources of such data can be census data, field trips, aerial photographs, telephone interviews, topographic maps, and any other source that gives a realistic estimate of flood plain occupancy and usage.
Step 6: Estimating fatalities rates from empirically based equations or methods.
Suggested rates for estimating life loss should be obtained from Table 6, which was developed using data obtained from approximately 40 floods, many of which were caused by dam failure. The 40 floods include nearly all U.S. dam failures causing 50 or more fatalities as well as other flood events that were selected in an attempt to cover a full range of flood severity and warning combinations. Values of fatality rates given by this table are subject to the following factors:

i. Flood Severity,
ii. Available warning time to population at risk,
iii. The degree of understanding of the flood severity by those issuing the warnings (refer to notes given above below Table 6).

Number of fatalities may be obtained by multiplying the applicable fatality rates obtained from Table 6 by the numbers of population at risk PAR for each scenario of dam failure, which has been obtained from the previous step 5.

Table 6: Recommended Fatality Rates for Estimation Loss of Life Resulting from Dam failure.

Flood Severity	**Warning Time (Minutes) see note (2)**	**Flood Severity Understanding See note 3**	**Fatality Rate**	
			Suggested	**Suggested Range**
High When flood sweeps the area clean. High flood should be used only for locations flooded by the near instantaneous failure of a concrete dam.	**No Warning**	**Not Applicable**	**0.75**	**0.3 – 1.00**
	15 to 60	Vague	Use the values shown above and apply to the number of people who remain in the dam failure flood plain after warning is issued. No guidance is provided on how many people will remain in the flood plain	
	More than 60	Precise		
Medium Homes are destroyed but trees or mangled homes remain for people to seek refuge in or on. Use medium flood severity if most structures would be exposed to depths more than 3.3 m or if DV* is more than 4.6 m^2/s	**No Warning**	**Not Applicable**	**0.15**	**0.03 to 0.35**
	15 to 60	Vague	0.04	0.01 to 0.08
		Precise	0.02	0.005 to 0.04
	More than 60	Vague	0.03	0.005 to 0.06
		Precise	0.01	0.002 to 0.02
Low No buildings are washed away off their foundations. Or most structures would be exposed to depths of less than 3.3 m **or** if **DV** (1) rate would be less than 4.6 m^2/s	**No Warning**	**Not Applicable**	**0.01**	**0.0 to 0.02**
	15 to 60	Vague	0.007	0.0 to 0.015
		Precise	0.002	0.0 to 0.004
	More than 60	Vague	0.0003	0.0 to 0.0006
		Precise	0.0002	0.0 to 0.0004

Notes:

1. The parameter DV is used to separate areas that might receive Low Severity Flooding from those areas expected to receive Medium Severity Flooding. DV is computed from:

$$\mathrm{DV} = \frac{\mathrm{Qdr}-\mathrm{Q2.33}}{\mathrm{Wdf}}$$

Q_{dr} is the peak discharge at a particular site caused by the flood wave.

$Q_{2.33}$ is the mean annual discharge at the same site. This discharge can be estimated, and it is an indicator of the safe channel capacity.

W_{df} is the maximum width of flooding caused by dam failure at the same site.

2. Warning Time: The warning time categories shown indicate the time interval from beginning of the flood wave at the dam break to the time people in downstream become aware of the coming flood. The fatality rate in areas with medium severity flooding should drop below that recommended in Table 6 if the warning time increases well beyond one hour. Repeated dam failure warnings, confirmed by visual images on television showing massive destruction in upstream areas, should provide convincing evidence to people that a truly dangerous situation exists and of their need to evacuate. This should result in higher evacuation rates in downstream areas and in a lowering of the fatality rate.
3. Flood Severity Understanding: The flood severity understanding categories are as follows:
    i. Vague Understanding of Flood Severity means the warning issuers have not yet seen an actual dam failure or do not comprehend the true magnitude of the flooding.
    ii. Precise Understanding of Flood Severity means the warning issuers have an excellent understanding of the flooding due to observations of the flooding made by themselves or others. Warning in case (ii) will be weak and the response of people will also become weak. In case (ii) the warning will be strong, and the response of people will be expected to be also become strong.

Step 7: Evaluating Uncertainty

Various types of uncertainty can influence loss of life estimates. To ensure more reliable results it is necessary to perform the following actions:

Action 1: The suggested procedure for this action is to develop separate loss of life estimates for each dam failure scenario. Various causes of dam failure will result in differences in downstream flooding and therefore result in differences in the number of people at risk as well as the severity of the flooding.

Action 2: It suggests that the dam failure be assumed to occur at various times of the day or week. It is recognized that the time of failure impacts both when a dam failure warning would be initiated as well as the number of people who would be at risk.

Action 3: It focuses on when a dam failure warning would begiven. This warning initiation time could be varied to determine the sensitivity to this assumption.

From the preceding it may be seen that dam failures can result in death of innocent people. In the dam failure records there are plenty of cases of such occurrences that could not be avoided. But, it is important to carefully examine case histories and infer new knowledge of the causes, and find ways to avoid such failures. Nobody would think that it is possible to eliminate the probability of failure and bring it to zero value. Therefore, it is most important to secure the downstream population to the maximum limit by estimating the dam hazards and implement procedures to reduce loss of life through;

i. Flood zoning of the potential inundation area,
ii. Installation of warning systems that can give enough warning, train dam tenders to be vigilant and observant and act in the proper time,
iii. Preparation of emergency evacuation plans ready to meet such events.

5. Historical Dam Failures and Their Consequences

Studying case histories of dam failures help to understand why dams have failed and to stress caution in dealing with existing dams in order to enhance their safety and reduce their hazards towards the downstream areas.

In an attempt to verify the conclusion of the previous chapters one sample of 25 dam failure cases is selected for examination.

The sample is compiled from historic dam failure cases, which resulted in a significant number of fatalities. ICOLD data base was utilized [15], additional information are added on the causes of failures. To make the size of the sample reasonable the selected cases fatalities ranged between 2200 and 100 in spite of the fact that hundreds or even thousands of cases have resulted in fatalities less than 100; while catastrophes like the case of 1975 failure of Banqiao and Shimantan Dams in China (about 23,000 deaths) are considered as special cases of multiple dam failures in Domino Action, and therefore, they are not included.

In the prepared Table 7, selected dams are arranged in descending order of fatality numbers. Date of failure is included to indicate the standard of knowledge or state of the art in dam design and construction during the time of failure. Causes of failure are listed in addition to the heights of the dams (H) and volumes of reservoirs (V). By inspecting this table it becomes clear that although the height (H) and volume (V) play prominent roles in causing fatalities, they are not the only effective factors. The failure of South Fork Dam in USA, which had 18 million cubic meters of storage, killed 2200 people, while Iruhaika Dam failure in Japan, which had the same storage, killed 1200 persons only.

Mode of failure can cause higher fatalities, as they tend to happen at faster rate than the other modes of failure as in case of overtopping during high floods and intensive storms.

Table 7: Dam Failure Cases, Cause of failure, Type of dam, Volume of reservoir and number of fatalities.

Dam	Country	Year of Failure	Cause of Failure	Type of Dam	Height of Dam (m)	Volume of Reservoir (hm^3)	Number of Fatalities
Vajont	Italy	1963	O	A	262	Not relevant	2400
South Fork	USA	1889	O	E/R	21	18	2200
Machu	India	1979	O	E	26	101	2000
Iruhaika	Japan	1868	-	E	28	18	1200
Möhne	Germany	1943	H	M	40	134	1200
Khadakwasala	India	1961	FL	M	33	137	1000
Tigra	India	1917	FL	M	25	124	1000
Panshet	India	1961	I	E	49	214	1000
Gleno	Italy	1923	O	E	35	5	600
Puentes	Spain	1802	O	M	69	13	600
St. Francis	USA	1928	S	GR	62	47	450
Malpasset	France	1954	F	A E	66	47	420
Dale Dyke	GB	1864	I	E	29	3.2	230
Sempor	Indonesia	1967	I	R	60	56	200
Fergoug 1	Algeria	1881	FL	A	33	30	200
Gotvan	Iran	1980	-	-	22	-	200
Vega de Terra	Spain	1987	O	B	33	7.3	140
Mill River	USA	1874	-	-	13	-	140
Hyogiri	Korea	1961	-	E	15	0.2	139
Walnut Grove	USA	1890	O	R	31	11	129
Kantilla	Siri Lanka	1986	O	E	27	135	127
Zerbino	Italy	1935	S	GR	16	10	100
Eder	Germany	1943	H	M	48	200	100
Nanak Sagar	India	1967	I	E	16	210	100
Heiwalke	Japan	1951	-	E	22	0,2	100

Nomenclature:

Cause of Failure	Internal Erosion	Spillway Defect	War Hostilities	Overtopping	Stability	Foundation	Flood
	I	S	H	O	S	F	FL

Type of Dam	Masonry	Earth Fill	Rock Fill	Multiple Arch	Arch	Gravity	Buttress
	M	E	R	MA	A	GR	B

The conclusions that may be drawn from dam failure studies may be summarized as follows;

i. Mode of failure, whether sudden or takes some time contribute to the number of fatalities.
ii. Population at Risk (PAR) is very important. High population in the flood plain exposed to the danger of drowning increases the number of fatalities.
iii. Warning the (PAR) in very short time or failing to do so increase the number of fatalities and vice versa.

The technical safety of the dam on which point (i) above depends is the responsibility of the dam owner and his personnel on site, but ensuring the safety of downstream population referred to in point (ii) and point (iii) is the duty and responsibility of the communal authorities and government departments of the area or zone in which the dam is located and, which falls under their jurisdiction. This duty encompasses such things as, flood zoning planning, preparation of Emergency action plans (EAP), and finally installing good alarm systems

6. Conclusions

Due to the increased public awareness of the hazards that could result from dams failures of old dams or even new ones, procedures and actions are needed to reduce the threats they cause to communities living in the downstream flood plains. Based on actual failure cases and the related number of casualties new dam safety classification systems are adopted; these are linked directly to the potential losses of lives and properties. It is also recognized that DAM SAFETY has two major issues; these are:

i. The safety of the dam structure itself,
ii. The safety of the population at risk (PAR).

The first issue; for new dams the construction of such dams should be allowed only after fulfilling all safety requirements of thorough geological and hydrological investigations and deep analytical studies, preparation of safe designs, and by following proper construction procedures, which are commensurate with the class of the hazard classification of the dam under consideration. Thorough knowledge of case histories of dam failures helps in avoiding previous mistakes and results in safer designs. For existing dams safe operation and routine safety inspections and maintenance must be observed and followed. In the case of any defect is being discovered then corrective actions should be performed. One lesson learned from some case histories is that budgeting issues should not compromise needed corrective actions. Prioritization of the required repairs may help but without undue neglect or delay even for small issues. Aging dams form other source of threats due to their degradation and state of their materials deterioration. Safety of these dams must be reviewed regularly, and all necessary repairs and corrective actions must be applied promptly, or if this appears, unpractical or not economical, then one single moment must not be wasted, and they should be decommissioned immediately. These technical and managerial decisions must be handled by competent staff should they be design engineers, operators, or dam owners.

In dealing with the second issue related to minimizing the dam safety hazards towards the (Population at Risk), no one would think that it is possible to eliminate the probability of failure and bring it to zero value in spite of all efforts in this respect. So, it follows that maximum effort must be directed towards reducing dam hazards towards this. Population through regional planning and flood zoning of the potential inundation areas, installation of efficient warning systems that can give enough warning time, increase public awareness of necessary actions in case of a catastrophe, employing trained and vigilant dam operators and to have one or many of them stationed at the site at all times, and above all to have at hand proper and workable emergency evacuation plans ready to meet such events. An example of such a successful plan was witnessed during the Oroville Dam crisis in 2017, when 200,000 people were evacuated [16].

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Dam Safety and Overtopping

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Abstract

Overtopping is one of the most serious modes of failures for all dams causing great numbers of human fatalities and material damages. Statistics show that overtopping failures are the highest, especially for embankment dams. The main reason for this is the erroneous prediction of the inflow design discharge, which has resulted from lack of realistic flow data and imperfect hydrological procedures. Failure in most cases occurs when the inflow exceeds the spillway design capacity, but to a lesser extent from the buildup of very high wave setup and runs up. This has led to active efforts in upgrading dams for such occurrences, by either upgrading spillways, adding auxiliary spillways, increasing freeboard by either heightening the dams or the parapet walls on the crest. Advancement in predicting the safe inflow discharges are also made by adopting such procedures as the calculation of the Probable maximum flood based on predicting the Maximum Probable Precipitation or using statistical methods by treating long records of available flow data. Recently, another challenge has come up facing dam owners and builders who are represented by the climate change impacts on the hydrological cycle; this has put a new responsibility to the governments to issue new regulations and plans to mitigate these impacts reducing failure possibilities and improve dam safety against overtopping.

Keywords: Overtopping, Inflow Design Discharge, Spillway Design Discharge, Wave Set Up, Wave Run Up, Probable Maximum Discharge, Probable Maximum Precipitation, Climate Change.

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1. Historical Overview

Overtopping of a concrete, masonry or an embankment dam is the flow of water over their crest leading in the majority of cases to failure. In many events, during the long world history of dams; overtopping has been one of the major modes of failure. Most failures were caused by floods, which had exceeded the spillway capacity as a result of insufficient knowledge of the hydrological conditions of the region leading to incorrect assessment of the proper inflow design discharge. Many other overtopping failures, however, have resulted from a variety of reasons; such as failures of gates to operate properly as a result of total or partial jamming due to mechanical or electrical problems or operational errors on the part of the operators, jamming of the outlet structures by excessive quantities of floating debris reducing the discharge capacity of the outlet, landslide in the upstream reservoirs (artificial or natural) causing a surge wave over the dam crest. Excessive settlement and loss of freeboard after earthquakes have contributed in some cases to overtopping failures while in some other events the action of very strong waves resulting from violent storms caused overtopping failures due to abrasive process causing the failure of insufficiently lined upstream faces of dams. Moreover, human failings may have been behind some failures due to not observing the rule curves and operation instruction due to insufficient expertise on the part of the operators. According to statistics related to overtopping failures in the USA which have resulted from inadequate spillway design, debris blockage of spillways, or settlement of the dam crest account for approximately 34% of all U.S.A. dam failures, while the Association of Dam Safety Officials (ASDSO) data base shows that failure cases due to overtopping in the USA for the period 2010- 2017 exceeded by far all the other failure mechanisms, as shown in Figure 1.

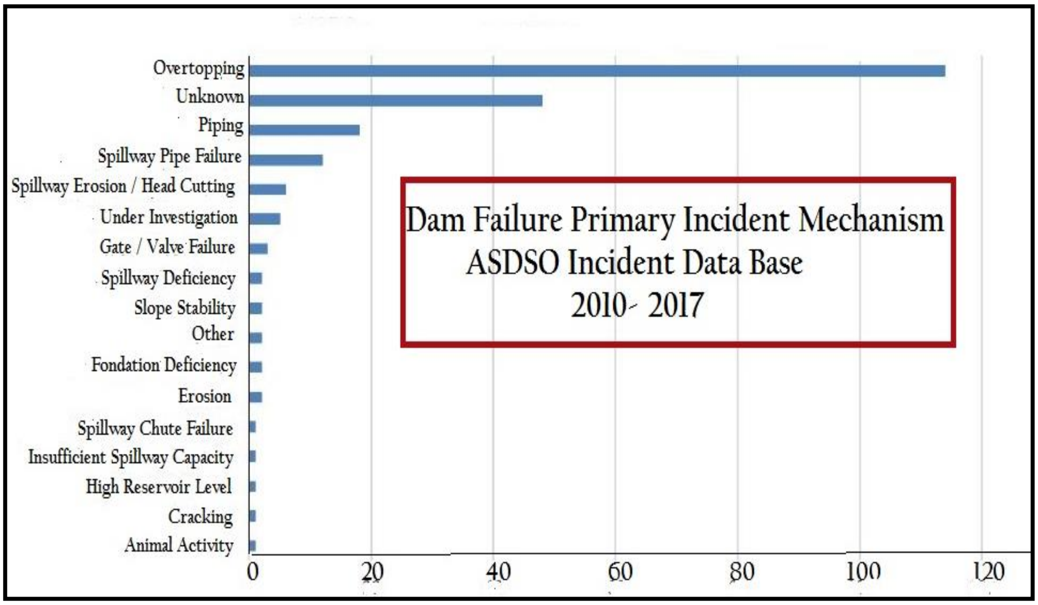


Figure 1: Overtopping vs. other Failure Mechanism (ASDSO), USA [1].

These failures have caused considerable damages and loss of life in the communities living in the downstream unaware of the danger to which they were subjected. Records indicate that only eight of the documented failures due to overtopping killed 8000 people during the period (1889-1951), while other statistics show a figure of human fatalities of almost 180,000 during the period (1951-2018) due to hydrological reasons mainly causing overtopping; out of these 171,000 people were drowned in the catastrophic failure of Banqiao and Shimantan cascade of dams in China in domino action in 1975 due to overtopping.

The high rate of fatalities due to hydrological reasons reflects that, even with improved hydrological methods and measurements, failures have continued to happen as a direct result of the high growth rate in dam construction and the sharp increase in their number during the last decades of the last century. To illustrate this point, Figure 2 gives the number of dams constructed in the world (by decades) up to the year 2000 according to their type, and Figure 3 indicates the number of dams constructed per given decade up to the year 2000 depending on dam height (hd). The source of these data is World Dam Register (ICOLD); in both cases not including dam failures in China and Russia [2].

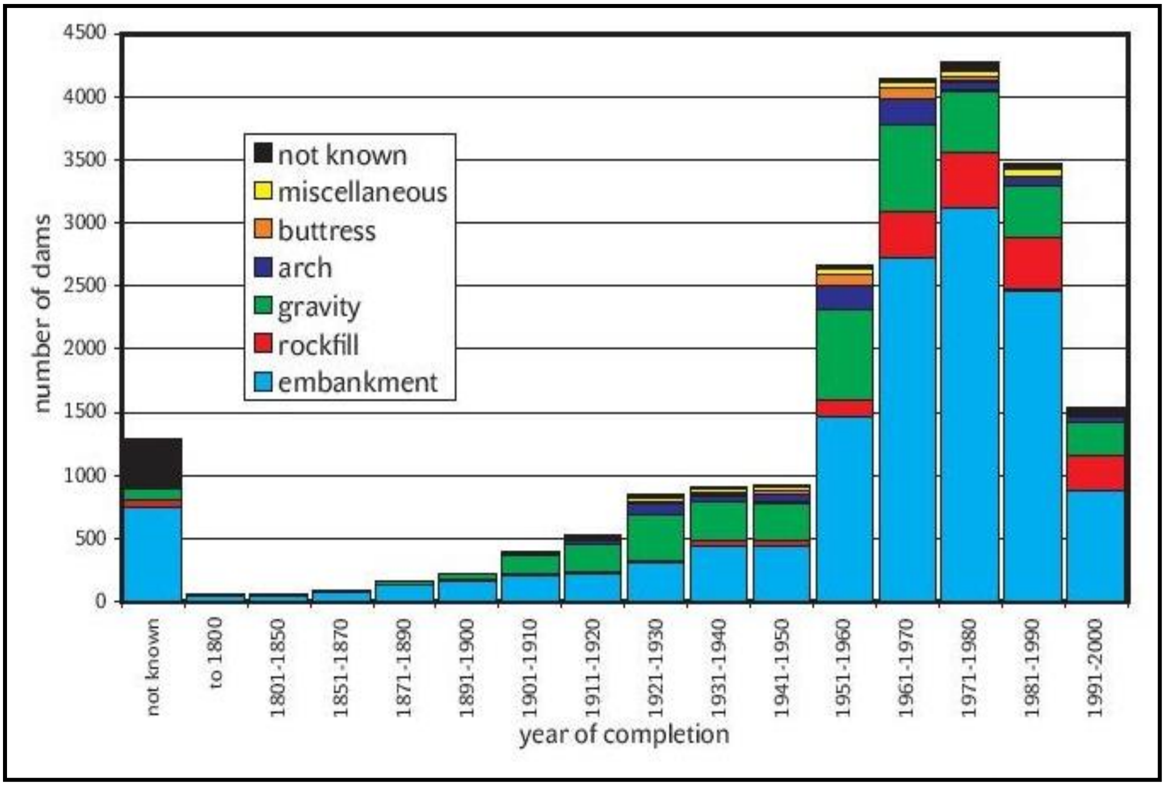


Figure 2 : Number of dams constructed in the world by decades and with reference to dam type, according to the World Dam Register (ICOLD) (not including dams in China and Russia) [2].

On a worldwide scale, floods have been the main cause of dam failures and of 90 per cent of the related fatalities. Most fatalities resulted from failures of 20 to 30m high dams, and few have resulted from thousands of failures of dams lower than 15m [3].

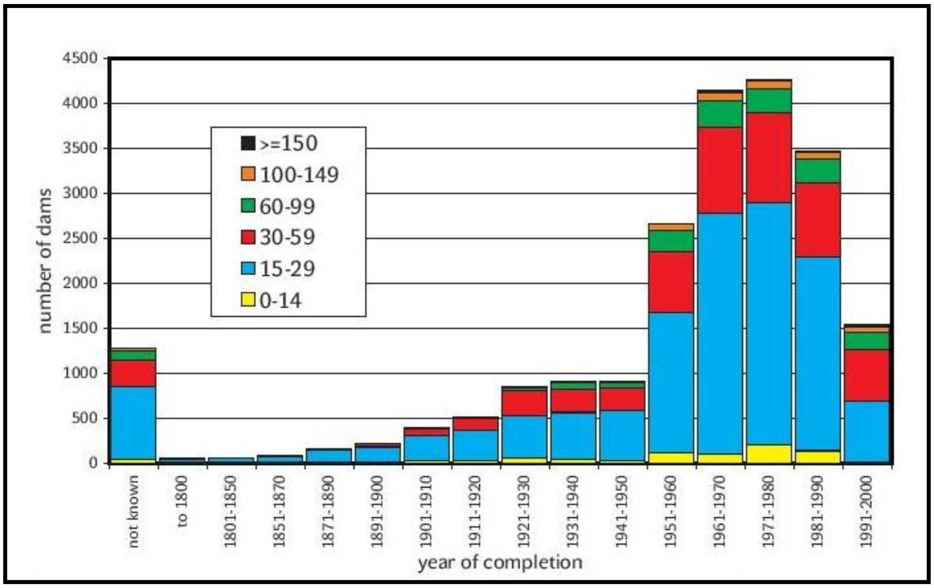


Figure 3: Number of dams constructed per given decade depending on dam height hd, according to World Dam Register (ICOLD) (not including dams in China and Russia) [2].

2. Definition of Terms

- **Overtopping**

According to *Collins English Dictionary* overtopping is: “The rising of water over the top of a barrier. In the context of dams; Dam overtopping means: water passing over the top of the dam which may lead to its failure unless it is designed for such an even. This condition arises when the passage provided to pass the flood (spillway and outlet works) are unable to discharge the coming inflow. Dams can be overtopped with a continuous flow when the Maximum Water Level of the reservoir exceeds the dam crest. For overtopping by waves, the water surface elevation approaches but does not exceed the crest level, but waves driven by wind produce waves run-up which overtop the top of the dam. The wave action can form an “equivalent” discharge per liner foot of the structure and can lead to the erosion and potential failure of the structure. Waves are influenced by wind speed, wind direction, bathymetry, open water distance, and embankment slopes.

- **Inflow Design Flood**

Inflow Design Flood (IDF) is the flood defined by the flow hydrograph entering the reservoir which is used to design and/or modify a specific dam and its appurtenant works. Particularly for sizing the spillway and outlet works, and for evaluating maximum storage of the reservoir, height of the dam and appurtenant works.

- **Spillway Design Flood.**

Spillway Design Flood (SDF) is defined by the flow hydrograph leaving the reservoir that is used to design and/or modify a specific dam and its appurtenant works (also referred to as the outflow hydrograph) the (SDF) is estimated by routing the appropriate (IDF) through the dam's spillway, outlet works, and attendant surcharge storage.

- **Deterministic Approach**

Deterministic approach as used in hydrology is the use of relationships in which the outcomes are precisely determined through known correlations between states and events, without any room for random variation. In this approach, a certain magnitude of a flood will always be the same for the same storm and for same catchment conditions.

- **Probabilistic Approach**

Probabilistic approach in hydrology enables variation and uncertainty to be quantified, mainly by using frequency distributions instead of fixed values in risk assessment. A frequency distribution describes the range of possible values (e.g. for floods), and shows the probability of exceedance of certain value.

- **Probable Maximum Flood (PMF)**

It is the flood that may be expected from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the drainage basin, and it is computed by using the Maximum Probable Storm, i.e. Probable Maximum Precipitation. Calculation of the probable maximum flood falls within the deterministic approach procedures.

- **Probable Maximum Precipitation (PMP)**

Theoretically, it is the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location during a certain time of the year. Calculation of the probably maximum precipitation falls within the deterministic approach procedures.

3. Detailed causes of dams' failures by overtopping

Overtopping leading to dam failures may occur due to variety of reasons as already mentioned; these which can be detailed as follows:

a. Inflow design discharge exceeds the capacity for which the spillway (together with other outlets if any) was designed and its capacity might have been underestimated in these cases. It has been noticed from reviewing dam failure cases due to overtopping that many of the older dams had failed as a result of the spillway being sized for floods that no longer represent the actual hydrological conditions. It is necessary; therefore, to carry out new evaluation of the older dams' safety by checking the inflow design discharge using updated hydrological data, and if the spillway capacity proves to be insufficient, this has to be rectified by either remodeling the spillway itself, add an auxiliary spillway or increase the height of the dam. In some cases, a combination of one or more of these solutions may prove more practical and cost effective.

b. Change of hydraulic conditions during the reservoir life time can have pronounced impact on the operation of the spillway leading to danger of overtopping by reducing the discharge capacity of the spillway from the previously calculated design discharge. Spillway discharge curves used in passing the floods during operation are often based on idealized discharge curves obtained from hydraulic modeling, and so it may be advisable; therefore, to check these curves immediately after commissioning and more than once during operation of the dam. Another factor which has its impact on the routing of floods is the possible changes that occurred in the flow conditions of approach due to natural or artificial changed throughout the life of the dam. New conditions which can arise that are less than the ideal assumed ones during design that may reduce the discharge of the spillway from what was originally stipulated. Moreover, sedimentation of the reservoir normally alters the stage-volume curve used in the flood routing equations to arrive at the size of the reservoir and the corresponding spillway design discharge during the design stage. These again warrants the revision and alteration of the operation rules for higher safety of operation. A good example of such change is illustrated in Figure 4 which shows the stage-volume curve of Mosul dam reservoir in 2011 after 25 years of operation (in red) as compared to the original curve (in blue). The red curve was obtained by an actual bathymetric survey, and it shows a considerable change. More expected changes are calculated for future periods (50yr, 75yr, 100yr and 125 yr) as shown in the other curves [4].

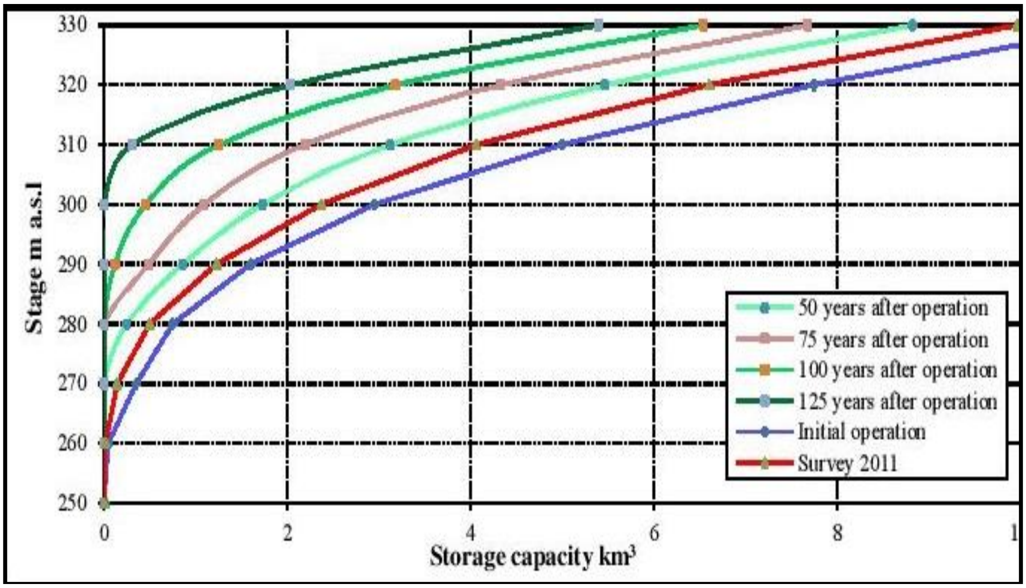


Figure 4: Storage- Discharge curve variation with time of Mosul Dam Reservoir.

c. During high floods spillway discharge capacity may be reduced due to malfunctioning of the spillway gates even at incoming flood, which is less than the inflow design discharge. A notable example of such an event is the overtopping and failure of Gibson`s dam in the USA when overtopping occurred during a rainstorm in Montana on June 6-8 1964. The spillway radial gates were not fully opened at the duration of the storm; two gates were completely open, but two more gates were partially open and the last two gates were completely closed. The controls of these gates were inaccessible to the operators during the storm; so overtopping of three feet of water over the parapet for twenty hours caused the collapse of this concrete dam [5]. More on this can be found in reference [6].

d. Overtopping and the consequent dam failure may result from a miss-operation of the spillway gates due to human negligence. The failure of the 24 meter high Dibis earth fill dam on the Lower Zab River in Iraq on the first of March 1984 can be mentioned as an example. In this event, the dam was left completely intendant and the operators had left the site during the night leaving no replacement to operate the spillway radial gates in case of emergency. These gates were partially open when the flood occurred, and high flow release from the upstream Dokan Dam combined with very high runoff from a severe rain storm on the intermediate catchment caused the overtopping and erosion of the earth fill embankment. The fuse-plug that should have worked in such rare occurrences did not erode due to complete short sight of the authorities who had constructed a concrete slab for passing a road on the crest and had laid also large diameter water supply pipe along it making the operation of the fuse plug impossible. Unofficial sources put the number of fatalities at nine.

e. Potential blocking of spillway entrance by reservoir debris may lead to catastrophic results. If the full capacity of the spillway is not available, dam overtopping can occur under more frequent floods. Some watersheds produce large amounts of debris during rainstorms. Sturdy log booms may be able to capture the debris before it reaches the spillway, but if not, the debris may clog the spillway openings. If a spillway is gated, and the gates are being operated under orifice conditions or if the bottom of the raised gate is less than 5 feet above the flow surface, then the spillway openings will be further restricted, compounding the potential for debris blockage.

f. It is interesting to note that overtopping can happen also from the combination of the still water level (SWL) and wind setup exceeding the crest of the dam. For over-wash, wind setup and wave run-up intermittently may combine to produce water level exceeding the crest of the dam. Typically, however, a significant surface area upstream (fetch) is required to allow wind to develop waves that would be directed towards the embankment and overtop it, Figure 5, [7].

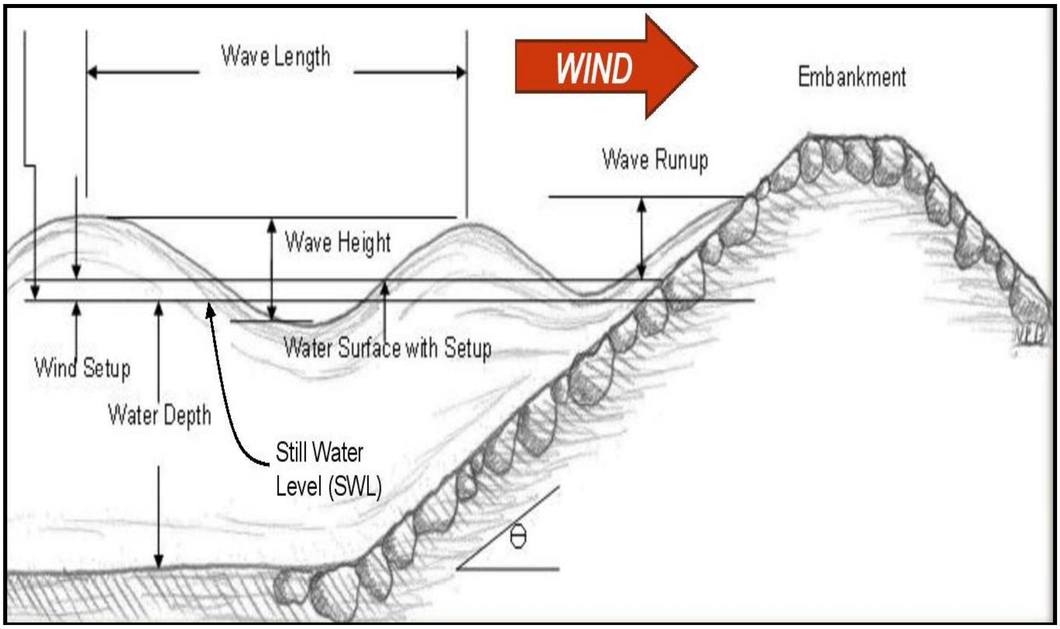


Figure 5: Definition of terms for wind action [6].

g. Other overtopping failures may be caused by landslides into the reservoir which cause a sudden surge in the upstream water level and formation of wave passing over the crest as it happened in the Vajont dam catastrophe in Italy. This dam was completed in 1959, but the landslide into the reservoir caused the overtopping of this concrete dam on the 9th of October 1963. Paradoxically, the dam did not fail but the flood wave caused the death of almost 2000 people. Earthquakes could also trigger landslides or may cause excessive settlement of the dam and loss of freeboard leading to overtopping.

4. Dams failure mechanism during overtopping

The Mechanism of failure caused by overtopping differs between earth fill dams and concrete dams. For earth fill dams, there are two ways for overtopping failure to be initiated. The first is when the flow over the crest of the dam continues as the reservoir water level exceeds the elevation of the dam crest, and second is the persistent over-wash from waves even if the water surface stays below the crest elevation. In such case the over-wash has to continue for enough time to erode the material of the crest and initiate the washing away of the dam body.

From past similar failures, it was noticeable that most embankment dams and levees would not withstand sustained overtopping of a foot to two feet or more without a high probability of failure. The depth and duration of overtopping are therefore, the main factors leading to failure. The embankment crest protection materials and the downstream protection materials contribute to the rate of erosion that leads in the end to failure.

The development of this process is initiated by a flood which causes the reservoir level to rise above the crest, then the slope protection is removed after reaching its critical shear stress and therefore, erosion of the body of the dam itself would begin by the erosion of the embankment. First along the top of the downstream slope of a cohesionless embankment, or at the downstream end of the crest of a cohesive embankment and headcut forms, as soil particles continue to be washed the headcut advances to the upstream end of the crest (and can deepen and widen at the same time) and eventually the embankment crest is lowered, and breach occurs leading to washing away the dam, refer Figure 6 and Figure 7 [7].

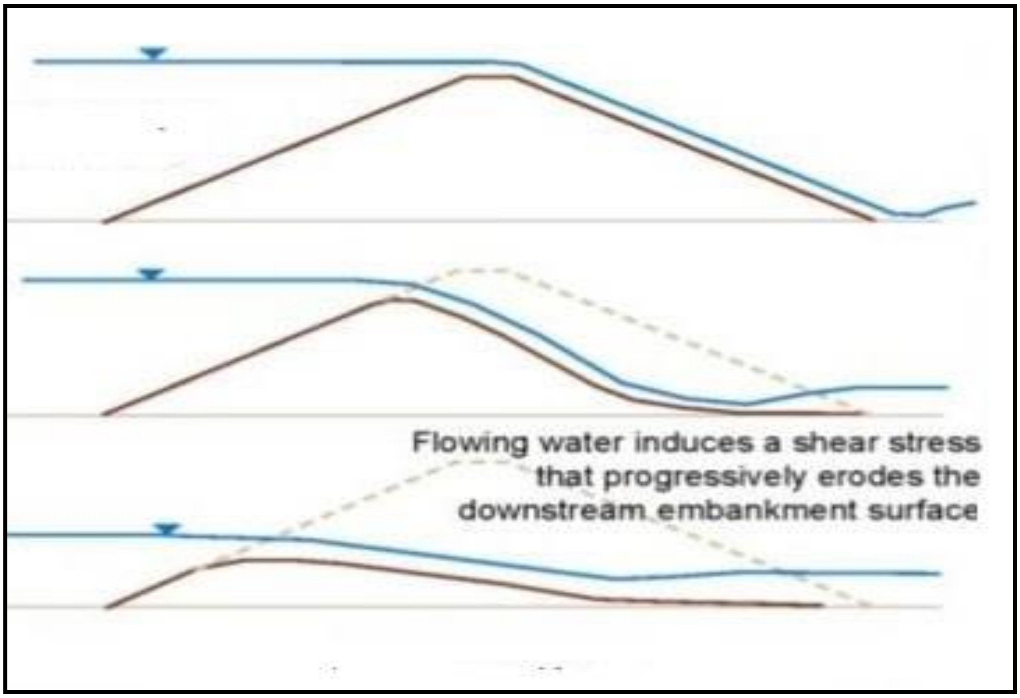

Flowing water induces a shear stress
that progressively erodes the
downstream embankment surface

Figure 6: Stages of failure of earth fill dam due to overtopping. (Cohesionless material) [7].

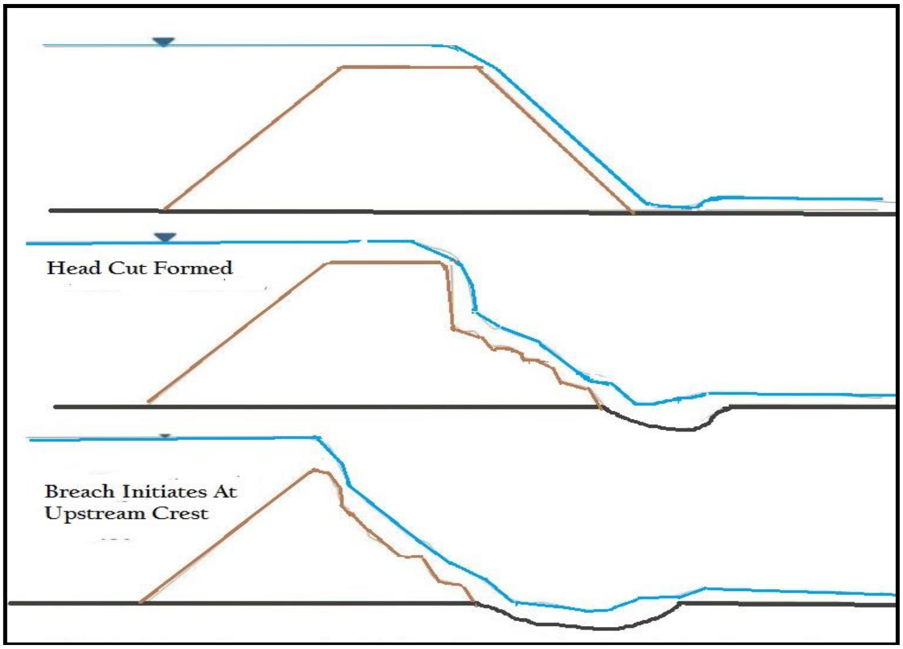


Figure7: Stages of failure of earth fill dam due to overtopping. (Cohesive material) [IV2].

In most cases, concrete dams can withstand certain level of overtopping more than what earth fill dam can do. This is due to the resistance offered by the dam material itself, but their vulnerability to failure may be due to scouring of their foundation rock at the downstream. The rock material and the condition of jointing and fracturing of the rock mass, their magnitude and spacing and bedding orientation are all factors which can enhance the erodibility of bed material in the vicinity of the dam foundation. In sustained conditions of overtopping, the depth of scour increases and could even scour the foundation leading to the collapse of the dam. The lowering of the Tailwater level as erosion downstream continuous enhances the scouring process leading to the mentioned conditions [5]. The dam usually collapses almost instantaneously, and a positive direct flood wave (shock wave) is formed. This wave distinguishes itself by a steep front advancing forwards as a result of the so-called “flood-gate effect”. Then, a backward negative wave spreading in the reservoir against the flow direction originates at the reservoir level of the broken dam. Maximum breach discharge practically occurs at the beginning of the failure with the maximum difference of water levels on the upstream and downstream sides of the dam [9]. The breach may be instantaneous, and its length can be about five times the dam height. As the reservoir level, which corresponding to the failure, may not be well known, alarms are less efficient in cases of concrete dams than for embankment dams, and old dams require great care, as the risk of fatalities is high [3].

5. Inflow Design Flood and Dams Safety

One of the most common causes of dam failures is the inability to safely pass flood flows. The most common potential failure modes associated with hydrologic conditions include overtopping erosion, erosion of spillways, internal erosion (seepage and piping) at high reservoir levels, and overstressing the structural components of the dam. Selection of the inflow designs flood (IDF) which is compatible with the required standard of safety requirements, therefore, is of utmost importance.

Inflows Design Flood (IDF), as already defined, is the flood used to design and/or modify a specific dam and its appurtenant works; particularly for sizing the spillway and outlet works, and for determining surcharge storage and height of dam requirements, in another word it is the largest flood that a given dam is designed to pass safely whether during construction or throughout its service life. It is, therefore, the largest flood that is selected for the design or for safety evaluation of the dam.

In selecting the inflow design flood (IDF), engineers had used a variety of methods to select this flood discharge using available hydrological data, and at the same time having in mind the size of dam and reservoir and their outlook towards safety requirements plus available financial resources. The used methods varied from the use of deterministic approach such as the use of rational formulae, which were widely used in early times and still being used in some countries and another popular approach, which is based on the unit hydrograph.

In the rational method, the peak runoff rate is a function of the average rainfall rate during the time of concentration such that and rainfall intensity is constant during rainfall, and is established in the relationship between the peak discharge and the design rainfall of the same frequency in the form of:

$Q = kCIA$ where,
k – Conversion factor equal to 1.008 (SI) or .00278 (metric)
C – Dimensionless runoff coefficient.
i – Rainfall intensity (in/hr, mm/hr).
A – Catchment area (acres, ha, km^2).

In using the unit hydrograph method, the design storm for the project is applied to determine the corresponding flood hydrograph. More often the Probable Maximum Flood is also utilized as another variation of these deterministic methods. When a sufficiently long flow series of discharges is available, the probabilistic approach is resorted to where frequency analysis is carried out to determine the floods of various frequencies or probabilities. For important projects, the results of various approaches are compared or combined together to obtain the design flood.

In examining the procedures followed so far, both the probabilistic and the deterministic criteria have been widely used. Probabilistic criteria are based on either floods or rainfall events, which have specified probabilities or return periods (such as the 1% annual chance exceedance flood), while the Deterministic criteria are based, among the other methods mentioned above, on estimates of the Probable Maximum Flood (PMF) from estimation of the Probable Maximum Perspiration (PMP) over the specified region.

More often, the use of composite criteria had been also done by prescribing an Inflow Design Flood from an equation that includes both a frequency event and some fraction of the Probable Maximum Event e.g. (50% Probable Maximum Flood + 100 year flood) or other combinations. Such combinations, however, are not favored by the United States Federal Emergency Management Agency (FEMA) on the ground that such guidelines are not linked to the hazards presented by these dams and the degree of risks they pose to life and material and environmental losses. Furthermore, FEMA recommends that the hazard potential classification system be used as the basis for IDF selection guidelines, and~~,~~ that the use of any classification system based on the size (height or storage volume) of a dam for IDF selection shall be discontinued. The hazard classification system which is specified by FEMA is illustrated by Table 1 [10].

Table 1: Recommended Dam Classification System Based on Hazard Potential [10].

Hazard Potential Classification	**Loss of Human Life**	**Economic Loss, Environmental Loss, and/or Disruption of lifeline Facilities**
High	Probable (one or more expected)	Yes (but not necessary for this classification)
Significant	None Expected	Yes
Low	None Expected	Low (generally limited to the Owner)

The use of FEMA classification system shown in Table 1 is based on the understanding that the failure of any dam, no matter how small, could represent a danger to downstream life and property. It is therefore, necessary that this classification be used as the basis for the inflow design flood selection, and that the use of any classification system based on the size (height or storage volume) of a dam for IDF selection is discontinued. Following this, Table 2 is recommended for use for the selection of IDF [11].

1) Incremental consequence analysis or risk-informed decision making may be used to evaluate the potential for selecting an IDF lower than the prescribed standard. An IDF less than the 0.2% annual chance exceedance flood (500-year flood) is not recommended.
2) Incremental consequence analysis or risk-informed decision making studies may be used to evaluate the potential for selecting an IDF lower than the prescribed standard. An IDF less than the 1% annual chance exceedance flood (100-year flood) is not recommended [11].

Whatever, the method used to arrive at the inflow design discharges the importance and reliability of hydrological data obtained from historical records should be tested for uniformity, reasonableness and reliability by means of theoretical computation and by comparative techniques against available data of precipitation and runoff records of neighbouring drainage area. If little or no historical records are available, data on which design can be based may be obtained from correlation with hydrologic and meteorologic records from neighbouring drainage areas. However, care must be exercised in determining relationships and conversion factors. The resulting values should be checked in the field by runoff measurements, comparison with marks left from floods, etc.

Table 2: FEMA's recommendation for the selection of IDF.

Hazard Potential Classification	Definition of Hazard Potential Classification	Inflow Design Flood (IDF)
High	Probable loss of life due to dam failure or misoperation (economic loss, environmental damage, or disruption of lifeline facilities may also be probable, but are not necessary for this classification)	PMF[1]
Significant	No probable loss of human life but can cause economic loss, environmental damage, or disruption of lifeline facilities due to dam failure or misoperation	0.1% Annual Chance Exceedance Flood (1000- year Flood)[2]
Low	No probable loss of human life and low economic and/or environmental losses due to dam failure or misoperation	1% Annual Chance Exceedance Flood (100- year Flood or a smaller Flood justified by rationale)

From the view point of preventing overtopping of any dam; the capacity of gated spillways should be sufficient to discharge the full inflow design flood (IDF) without taking into account the damping effect resulting from flood routing through the reservoir, and in determining or checking the spillway capacity no discharge from other outlet facilities should be taken into consideration. A reduction of the spillway design discharge may be permitted if:

1) Using higher criteria of safety against failure from overtopping than required by current design practice and technology or prevalent design standards.
2) Permanent availability of reserve storage capacity of the reservoir, between the normal and the maximum reservoir. The availability of the reserve

storage capacity must be combined with highly reliable operating procedures that assure the opening of the spillway gates in accordance with the predetermined flood routing program. When establishing this program, the risk of repetition of major floods at short intervals should be taken into account.

3) The existence of an additional fuse plug spillway; the rupture of which would not increase the downstream flood beyond an acceptable level.
4) A permanent low downstream risk level that should at no time includes any additional risk to human life in inhabited areas due to failure of the dam. In case of an increase of the downstream risk level, the spillway capacity must be re-evaluated and enlarged if necessary.
5) Other favorable circumstances that permit exemption from the above mentioned requirements, such as the existence of regulation put by the responsible government agency, as in taking into account the higher resistance against erosion of concrete dams as compared to earth dams; or the combination of a gated with an overflow spillways [12].

As outlined in (2) above, this point emphasizes the importance and necessity of reliable and intelligent operational procedures to alleviate the overtopping risk, even if all design requirements are satisfied. Moreover, the risk to life is taken up clearly in these recommendations. The upgrading of the spillway or the dam as a whole to safe guard against overtopping and the consequential loss of life and extensive property damage is clearly stipulated in point (4) above.

6. Overtopping Dam Failures; Some Examples

The possibility of overtopping of a dam, from hydrologic and hydraulic points of view, is governed by two important design factors; which are: 1) The selection of the Inflow Design Discharge (IDF), and 2) The selection of the Spillway Design discharge (SDF).

During the nineteenth century, or even before that, the flood inflow design flood discharge (IDF), required for the design of the early dams was entirely done by arbitrary methods, based on some empirical techniques, use of high water marks left by previous high floods and engineering judgment. This meant, the high possibility of overtopping of dams that were not built to meet unanticipated higher floods than those which they were designed for. One example of such dams in the United States is the Mill River Dam which failed on May 16, 1874, killing 139 people during a heavy storm when the reservoir was already full [13].

The second example of such catastrophe is the failure of the South Fork Dam which occurred on May 31, 1889, which had resulted from overtopping during a large storm event killing 2200 people creating what was known then as the “Johnstown Flood” [14].

Another classical case of dam failure due to miscalculating the inflow design flood and the inadequacy of spillway was the failure of the Walnut Grove Dam. In

February 1890, a combination of three days of heavy rain plus snow melts runoff filled the reservoir behind the South Fork Dam and threatened its collapse. The dam's biggest fault probably was an inadequate emergency spillway meant to keep the dam from being topped. The fate of the dam was sealed when the owner had scrapped one spillway plan in favor of a cheaper one right next to the dam which proved to be far too small. The reservoir filled up during a heavy rainfall storm on March 14-17, 1889. A minor enlargement of the spillway had been ordered in December 1889, but the hydraulic engineer confirmed that the spillway was still too small. For six hours, water topped the 400-foot length of the dam eroding the unprotected side. Workmen tried to widen the spillway in the midst of the driving rain, but it was too late. In the aftermath of the failure, 4 billion gallons of water poured into a canyon above Wickenburg and killed approximately 100-150 people, although no one will really ever know how many people drowned, [15].

On the continent, similar failures did occur, one example was the Sella Zerbino secondary dam on the Orba River in Italy. This dam was completed in 1935, and it was a straight concrete gravity dam which had a height of 46 feet and a crest length of 360 feet, Figure 8. This dam was a secondary saddle dam which together with the Bric Zerbino main dam formed the Orteglieto reservoir with a total storage of 14,600 acre- ft (18 million m^3). Depending on the discharge capacity of the main gated chute spillway and a pressure valved outlet of the main Bric Zerbino Dam no spillway was provided for the Sella Zerbino secondary dam. On the 13th of August 1935, the dam failed after several hours of very heavy rain, which had generated inflow discharge of about 100,000 cfs far exceeding the combined outflow discharge capacity of the main dam which was only 31,000 cfs that was based on very limited rainfall data, which may have corresponded to only 20 year discharge.

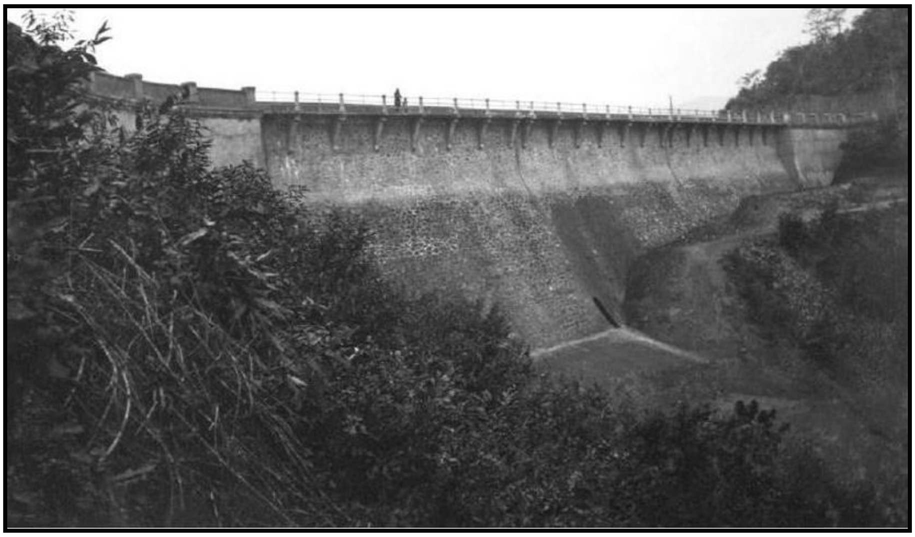


Figure 8: Downstream face of Sella Zerbino secondary dam

The apparent mode of failure was overtopping, resulting in erosion of the rock at the toe of the dam, in turn causing, sliding and overturning failure. As further evidence of the low quality of the geologic formation, little remained of the secondary dam and its foundation after failure, and geologic investigations after the failure indicated that while the main dam was found on compact serpentinites, the secondary dam was found on highly- jointed schistose rock with intense foliation and slaty cleavage, which many experts concluded were inadequate for dam foundation Figure 9.

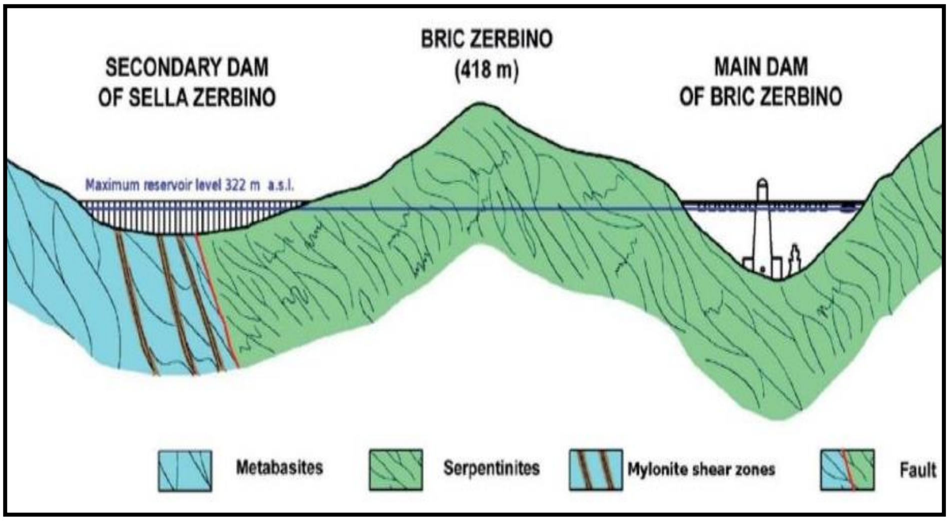


Figure 9: Geological section developed After the Dam failure.

The total losses suffered from this failure were at least 111 fatalities, destruction of a hydroelectric plant about 2 miles downstream, destruction of about 90 houses and 4 bridges and leaving a large impact on the downstream farming area which extended for 30 miles in the downstream basin [16], [17]. Overtopping failures of dams due to intensive unexpected rainstorms and resulting floods continued to happen in spite of the advances achieved in hydrological and meteorological sciences. One of the such major catastrophes associated with such an event occurred on the 11th of August 1979 when Machhu Dam II in Juagrat State in India failed. The dam was located on River Machhu near the town of Morbi and it was completed in August 1972. The dam had a storage capacity of 1.1 × $10^8 m^3$ and a height of 22.56m above the river bed, 164.5m of crest length of overflow section, and a total of 3742m of crest length for the earth dam. The spillway capacity provided was 5663m^3/s [18].

As a result of a heavy monsoon storm which had started on August 10, 1979 the flow began to increase down to the Machhu River, first hitting Machhu Dam I and then turning to Machhu Dam II. As the storm intensified, operators at Machhu Dam II began to open the gates to keep the water from rising above maximum levels. By 1:30 AM, all the gates were opened fully except for three gates that were not properly functioning. Despite the nonoperational gates, the dam was passing 196,000cfs, very close to its full capacity of 200,000. It wasn't enough, and the water continued to rise. It was early afternoon on August 11, 1979 when water overtopped the earthen embankment on both sides of the masonry spillway leading to the failure of Machhu Dam II. Approximately 90,000 acre-feet of water joined the already heavy flow from the river and rushed into the small town of Lilapar completely inundating it but luckily the population had already been warned and evacuated. Other towns and communities such as Morbi and Vajepar were not so lucky, and the death toll rose dramatically. No one knew for sure how many lives were lost, but estimates ranged from 1,800 to as high as 25,000. Part of the reason the number varied so much was that large mass graves were burned to keep diseases from spreading before proper records or any identification could be completed [19] Examination of rainfall data of past 80 years of stations in and around Machhu basin revealed:

1) There have been instances when stations in this region received their respective mean annual rainfall just in a single day;
2) Maximum 1-day rainfall at these stations varied from 180 to 510mm and 2-day from 330 to 680mm and,
3) PMP of 1-day duration worked out to be of the order of 450-1060mm. During the period 10-12 August 1979, Machhu basin up to Machhu Dam II received about 4 times the normal basin rainfall for August. Depth-duration analysis of past rainstorms over this basin revealed that the July 1894 rainstorm gave maximum rain depths for durations of 1 and 2 days while the 10-12 August 1979 rainstorm gave maximum rain depths for 3-day duration. Depth-area-duration analysis of the past severe rainstorms of this region has shown that, by and large, the 1927 rainstorm was the most severe and the rain depths measured in this storm were much higher than the rainstorm of 10-12 August 1979. Apparently, the large volume of water generated during the rainstorm of August 1979 was mainly due to the favorable antecedent basin conditions of moderate to heavy rainfall during the 10 days before the dam failure [20].

A review summary of failed dams in Gujarat State indicated the mistakes committed by civil engineers (!) in the estimation of the Spillway Design Flood (SDF) as given in the following Table (3):

Table 3: Design Floods, Observed, Highest Floods and Revised Spillways Floods for Some Projects, Gujarat, India.

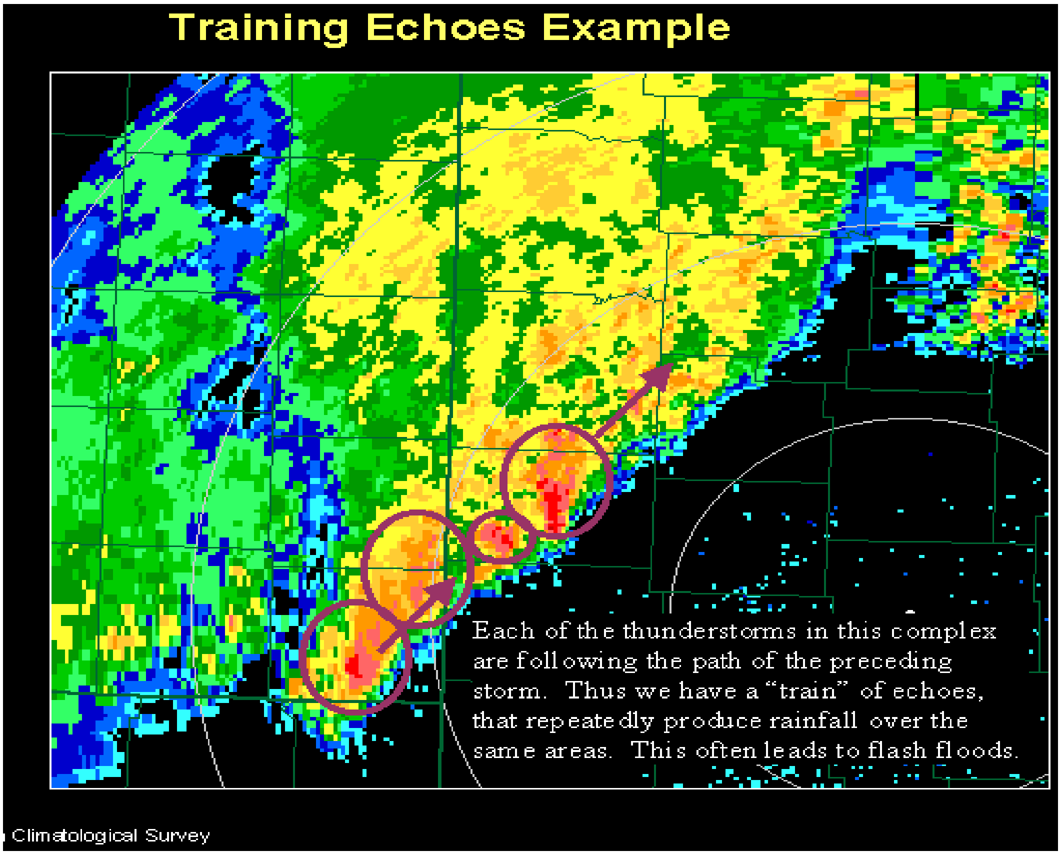
River Valley Projects in Gujarat	Total Catchment Area (sq. km)	Spillway Design Flood As Per Project Report (cumecs)	Highest Observed Flood (cumecs)	Revised Spillway Flood (Cumecs)
Dharoi	5486.00	11213.00	14150.00	21662.00
Dantwada	2862.00	6654.00	11950.00	18123.00
Machhu- 1	735.00	3313.00	9340.00	5947.00
Machhu- II	1928.00	5663.00	16307.00	20925.00
Damanganga	1813.00	11100.00	12900.00	12854.00

The above table shows the extent of error in the calculated Spillway Design Discharge and the required discharge leading to overtopping failure, which is the direct result of the inability to predict the safe discharge from available meteorological data [21].

Although dam engineering and hydrological science have improved over the past sixty or seventy years and the understanding of the risks and liabilities associated with the storage of water have increased also, most of this science and understanding are intuitive, comprehensible and accepted by dam owners and designers, but some of it remains not being so and is more mysterious, such as the size of potential extreme rainfall events and the resulting flooding that follows. Often combined effect of a series of storms moving over the same area, dumping heavy rain over several days can cause rainfall totals similar to a single extreme rainfall. Meteorologists refer to this as “storm training”.

It is such “storm training” events that dam designers must, in their designs, guard against and not only against overtopping failures by floods exceeding what is expected from a single extreme rainstorm [22].

Figure 10 shows radar echoes of a series of thunderstorms following the same path one after the other which can cause flash floods.



Each of the thunderstorms in this complex are following the path of the preceding storm. Thus we have a "train" of echoes, that repeatedly produce rainfall over the same areas. This often leads to flash floods.

Figure 10: Example of training echoes of a series of thunderstorms.

7. Overtopping threat and Dams Upgrading

Rehabilitation of dams is required to counter various deficiencies, which develop with time, and also to correct inadequacies on account of revisions of various standards/guidelines.

Deficiencies that are caused primarily by the ageing of a dam include degradation caused by weathering, wear and tear of equipment, damage from natural events, including floods, earthquakes or landslides, and damage from vandalism and war… etc.

Reviewing dams' safety, especially for older dam, includes among other things the review of the design criteria, assumptions and studies made during the planning and design stage such as those for the inflow design flood and spillway design flood.

The question of safety against overtopping stems from spillway adequacy, which is derived from the fact that new methods of analysis and new hydrometeorological data may indicate the use of a larger spillway design flood than the one used in the original design.

As hydrological data records accumulate and new methods are developed for flood discharge estimation and as higher requirements from the society regarding safety issues are expected, a large number of existing dams are in need for safety review, which may require, among other things, spillway rehabilitation, to improve their flood discharge capacity.

The importance of such a review is supported by many failures of dams caused by unrealistic estimation of these floods and the under sizing of the spillways and outlet structures which have led to overtopping of these dams. Governments and dams' owners, under the new safety regulations and legislations, are therefore, giving the upgrading of their dams a new look for better safety standards.

One way of increasing dam safety against overtopping, is to raise the dam itself and increase the reservoir volume to improve flood control operations, while a second option may be to modify the outlet works, especially the spillway. Adding another spillway may also be considered in other cases. Brief accounts of some examples of dam upgrading are given hereunder.

7.1 The Katsurazawa Dam- Japan;

Constructed in the upper reaches of the Ikushunbetsu River was completed in 1957. The height of the dam is currently being raised by 11.9m to improve its flood control capacity, irrigation and other functions. It will be reopened as the New Katsurazawa dam. The dam is 63.6m high and 334.3m long concrete gravity dam. Raising the dam crest was under way during 2018, and Figure 11 shows the dam during this stage of construction clearly indicating the extra height being added, while Figure 12 shows the dam after completing the raising operations [23], [24].

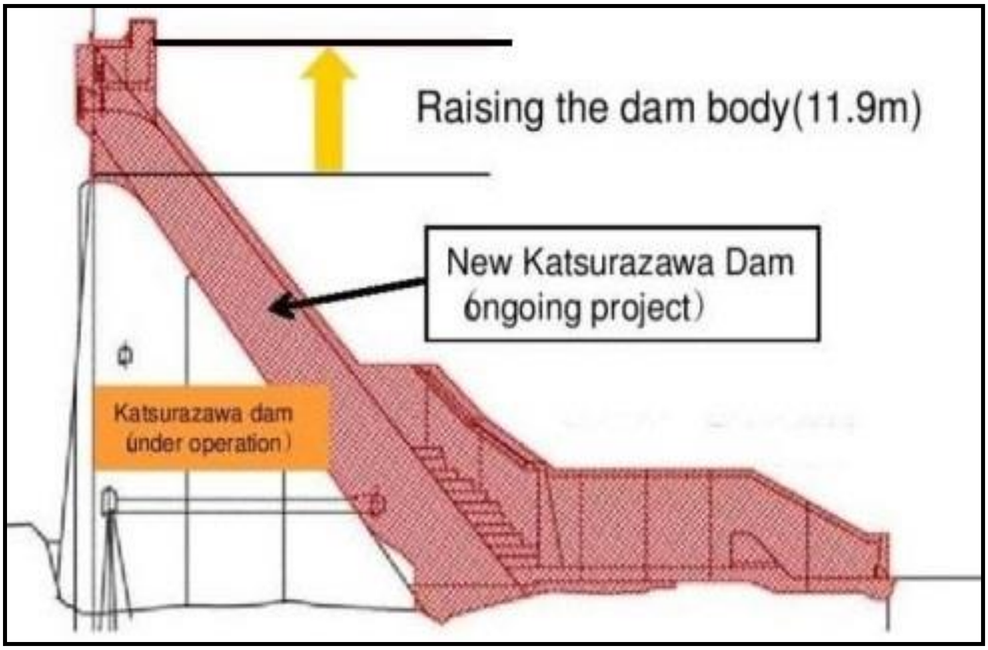


Figure 11: New Katsurazawa Dam (Major Heightening of Dam) [23].

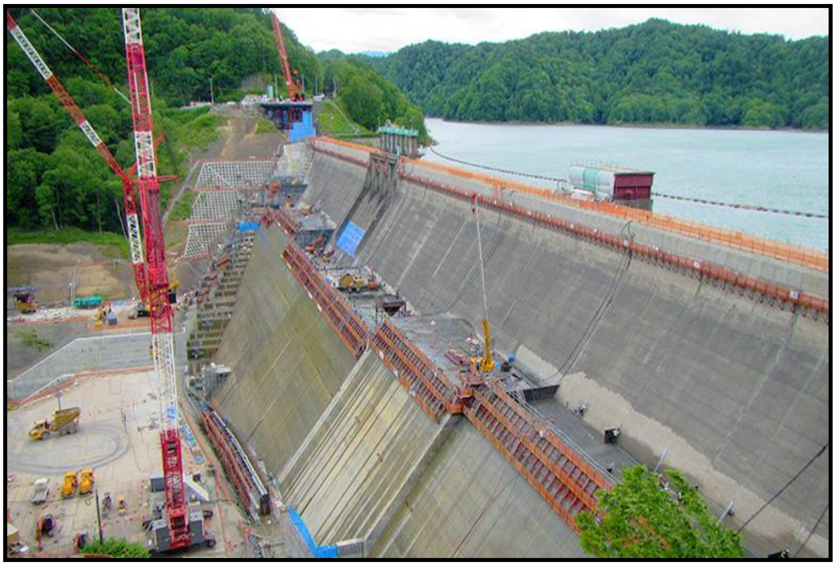


Figure 12: New Katsurazawa Dam during heightening operation in 2018 [24].

7.2 Kasabori Dam-Japan;

On the other hand, is a concrete gravity dam completed in 1964 to serve flood control, fresh water supply and hydroelectric power generation. The height of the dam is 74.5m and has a crest length of 225.5m, and the reservoir's capacity is 15,4 million m^3. This dam has already been raised by 4m to be 78.5m high to improve its performance and increase its safety, Figure 13 and Figure 14 show this dam after increasing its height [23], [25], [26].

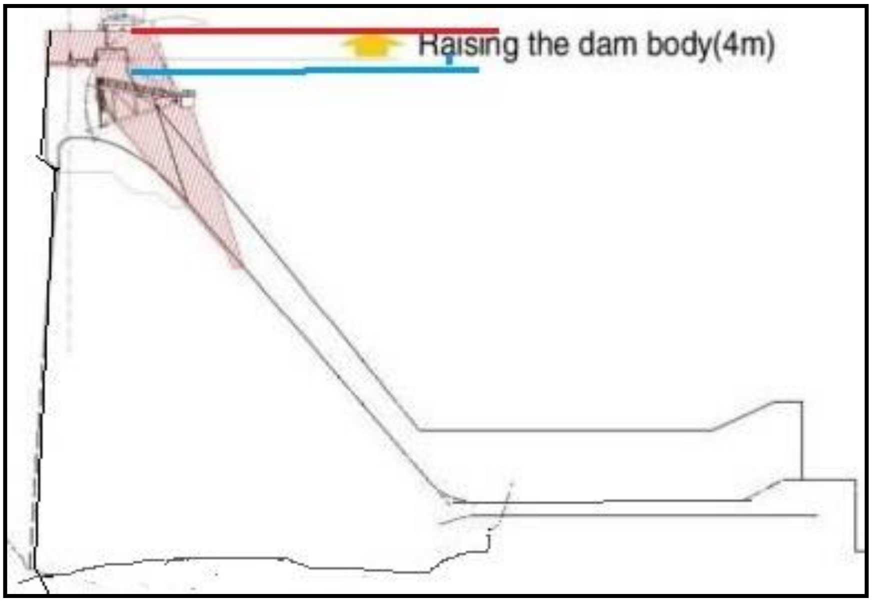


Figure 13: Kasabori Dam (minor raising) [23].

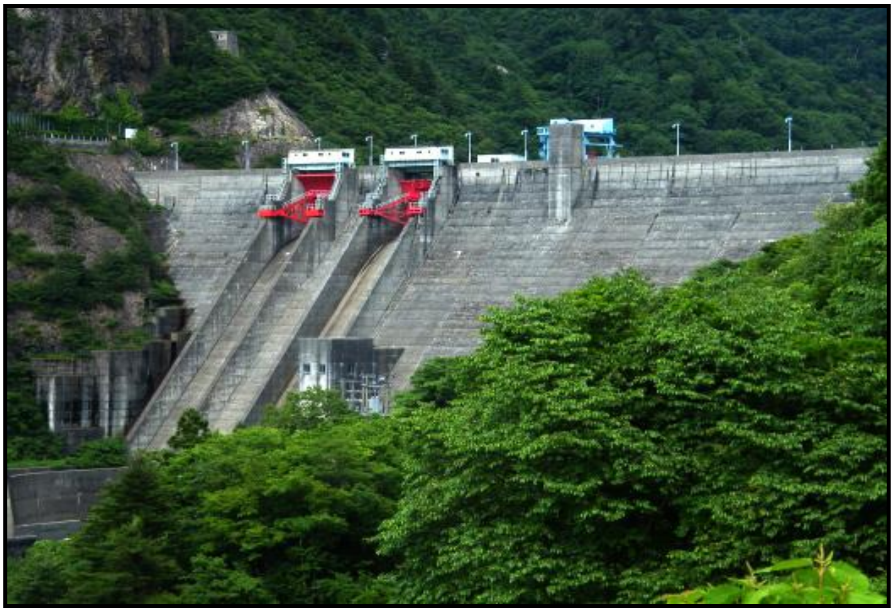


Figure 14: Kasabori Dam before heightening-[25].

7.3 Clarrie Hall Dam-NSW

This example is from New South Wales, which is the 45m high Clarrie Hall rockfill dam completed in the early 1980s. This dam was planned and designed in compliance with the engineering standard of the day. Since the dam was constructed, however, the ability to predict rainfall and understand extreme weather patterns have improved. The revision of the dam safety according to the new requirements of the NSW Dam Safety Committee showed that, although the dam's spillway can pass safely floods many times greater than the largest flood on record, it will, however, not be able to do the same in extreme rainfall events, even that the probability of such events may be very small. The upgrading works, which were to be completed in 2018, consisted of widening and extending the entrance of the existing spillway chute, constructing a parapet wall across the dam crest to increase freeboard and prevent water from flowing over the dam in an extreme flood event and enable the water to be released safely through the spillway, refer Figure 15 [27].

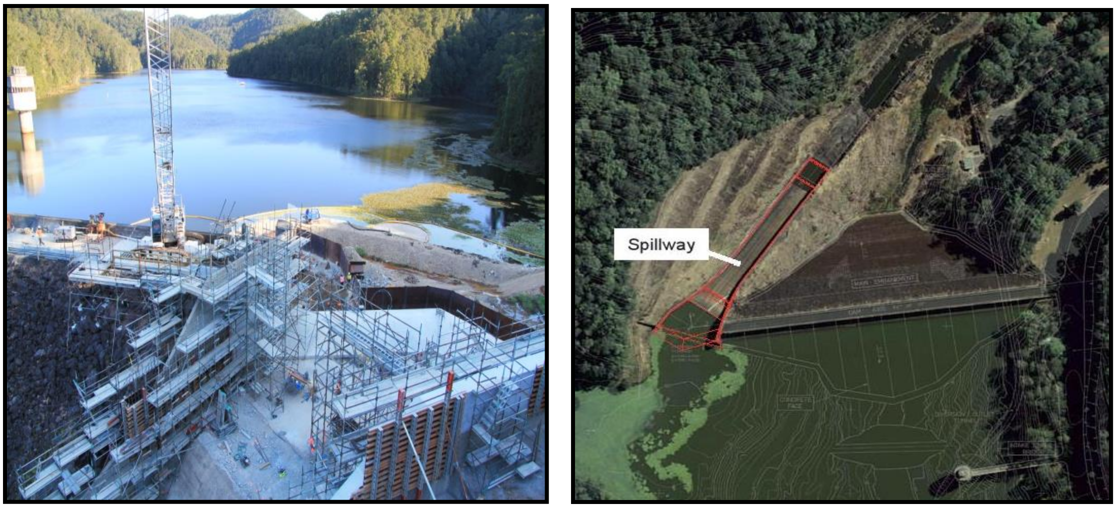


Figure 15: Clarrie Hall Dam (Left) aerial view showing spillway and embankment; (Right) spillway upgrading work.

7.4 Gloriettes Dam-France

This dam was built in the French Pyrenees Mountains, which is 40 m high concrete arch structure, Figure 16 and Figure 17. The dam was completed in 1951 and it had then a flood discharge system consisting of four over-flow sluices on the dam crest at elevation 1667m. Its capacity was $70m^3/s$ at a maximum operating water level at elevation 1667.8m. As hydrological data records increased and new methods for flood discharge estimation were developed it became necessary to increase the spillway capacity to meet the new standards of dam safety with respect to the risks to society. A new design discharge flood with 1000 year return period and peak discharge of $150m^3/s$ was defined. To compensate for the deficit of $80m^3/s$, a complementary new spillway on the right bank was to be constructed. For this purpose, a labyrinth weir, known as the Piano Key Weir (PKW) was selected. For the design of the chute of this new spillway, a solution was adopted after carrying out hydraulic modeling and it consisted of two stepped reaches and an intermediate stilling basin allowing 120° change of direction in order to accommodate the whole chute in the narrow gorge downstream [28].

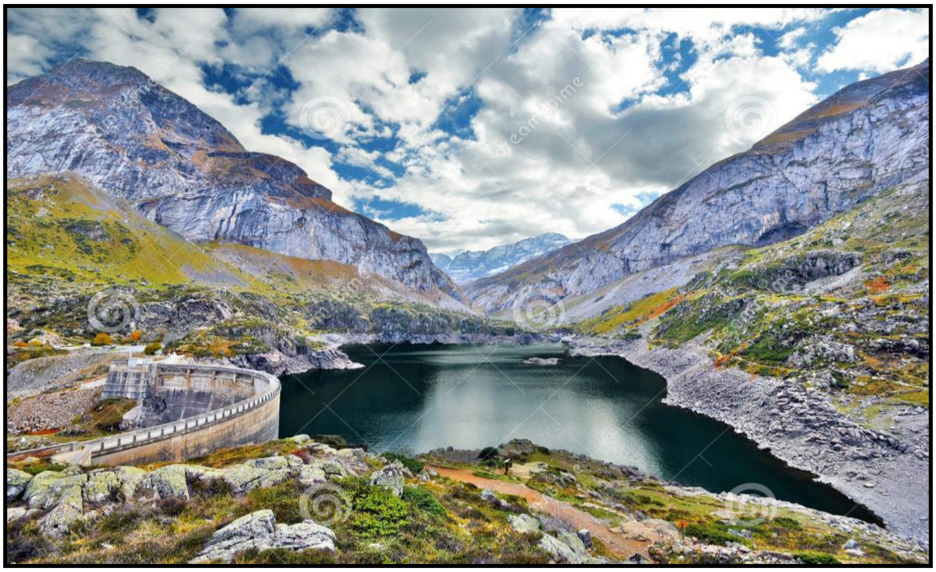


Figure 16: Gloriettes Dam upstream view.

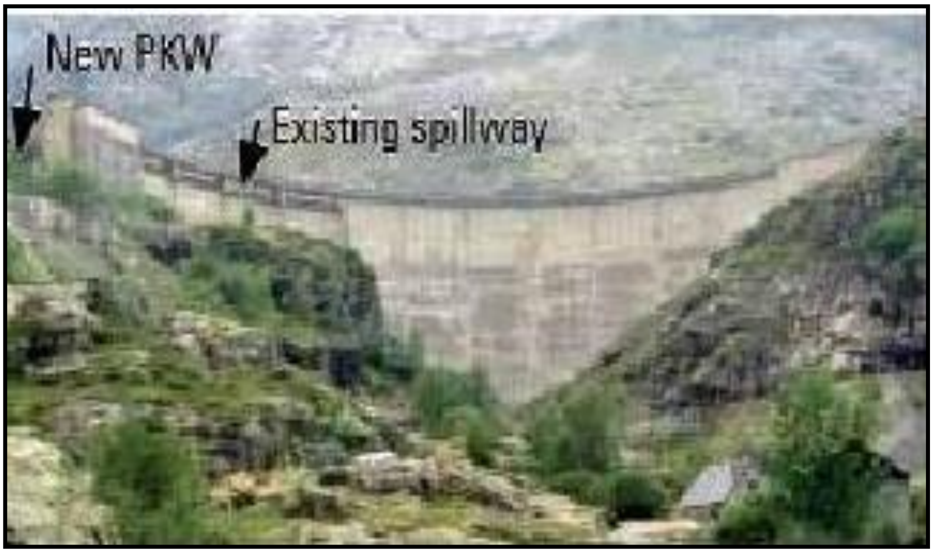


Figure 17: Gloriettes Dam downstream view showing location of the new Piano Key Weir spillway.

7.5 Nagayasuguchi dam-Japan

This is a concrete gravity dam which was completed in 1955 to serve power generation and flood control in Tokushima prefect in Japan. The dam has a dam crest of 200m length, height of 85.5m and a reservoir volume of 54,278,000m^3. The spillway was enlarged to increase its discharge capacity by adding two additional new bays with crest gates, as shown in Figure 18 [23].

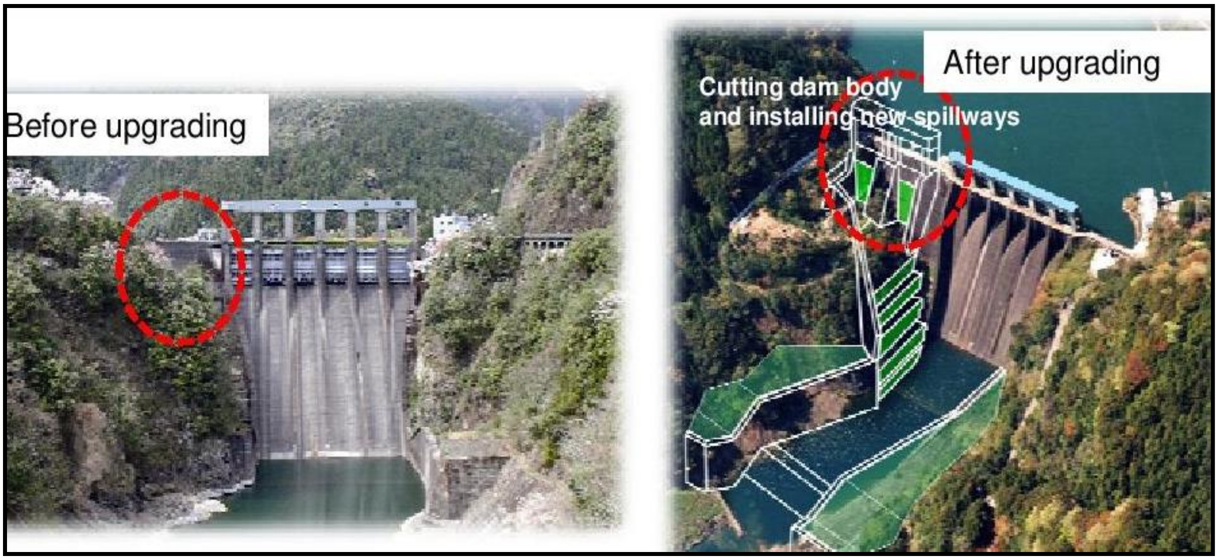


Figure 18: Nagayasuguchi dam before and after upgrading [23].

7.6 Höljes Dam- Sweden

This dam is an embankment dam serving power generation with hydraulic head of 88 m and a spillway on the right side. The dam was constructed on the Klarälven River in the south of Sweden and commissioned in 1962. In that era, dam safety was not a subject of state regulatory surveillance; there were no national directives that governed the dam design, construction, and supervision. The responsibility rested totally with the dam developer. The design flood of a dam was often finalized by multiplying the highest historical flood with a safety factor. For some small or even medium dams, the frequency analysis method was used, in which the 1000-year flood obtained through extrapolation, was treated as the design flood. The spillways were sized according to the dam-safety guidelines that were current at the time of construction, and the hydrological methods previously used to determine the design floods were inaccurate [29].

Based on the new flood determination criteria recommended by the revised dam-safety guidelines, many existing dams are found to have higher design floods than constructed for. The recent revision of the flood criteria was made in 2015. Among other dams the Höljes Dam spillway design capacity was increased to accommodate the increased revised discharge. The final solution is a new 17m wide spillway opening with an upward-going radial gate. The abandoned timber flume located to the right of the existing openings was removed to give place to the new gate, the width of the spillway channel downstream was doubled, and the stilling basin was also enlarged. These works were completed in 2016. Figure 19 shows the dam's spillway before and after rehabilitation [29].

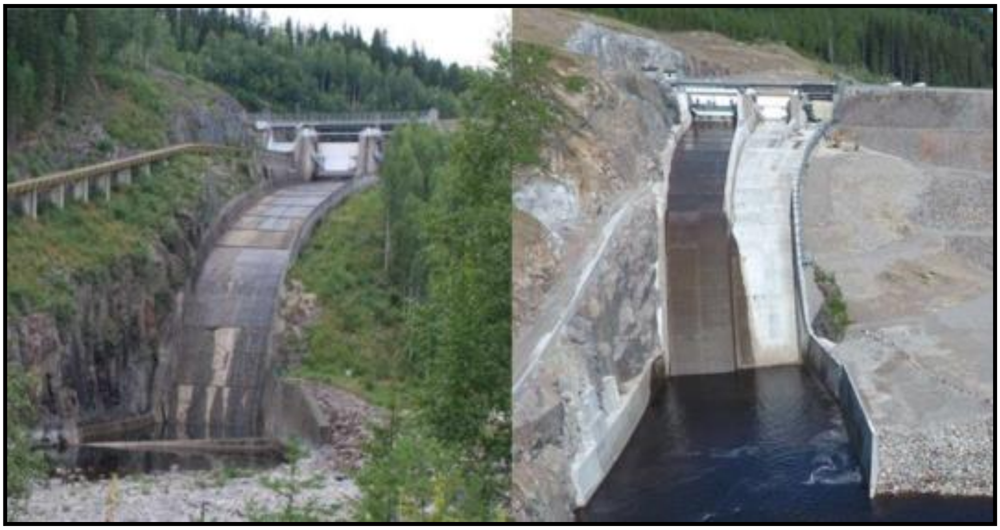


Figure 19: Höljes dam spillway: (Left) Original layout with two gated openings and a timber flume (Right) After refurbishment, with a 17m new gated opening added adjacent to the existing ones.

7.7 Usk Dam-UK

This is a 31m high, 480m long embankment dam with a clay core, which was constructed in 1955. The dam impounds a reservoir of 12.3Mm^3 capacity and located in Beacons National Park in South Wales. The reservoir is fed by both direct and indirect catchments and thereafter supplies raw water to Portis and Bryngwyn Water Treatment Works. In 2009, following completion of ongoing research into extreme rainfall, an inspection of Usk Reservoir recommended that an updated flood study be undertaken for the reservoir. In January 2014, a high-level leak was identified at the site, located within natural ground on the right abutment below the spillway. The leak was probably triggered by high rainfall and reservoir levels over the previous month. A dam safety study was carried out by an All Reservoir Panel Engineer (ARPE) and on the recommendation of the Panel, the reservoir has been held at a maximum of 1m below top water level (TWL) since that time. Pre-feasibility studies were completed in 2014, including a flood study and interim assessment of works, and hydraulic modeling. In 2016/2017, a feasibility study to investigate options for upgrading the Usk spillway was done. Many structural modifications were recommended including the existing overflow arrangement which was under capacity; flow was thrown out of the channel by existing baffle blocks, and the downstream stilling basin was largely ineffective. The existing wave wall was structurally deficient for the anticipated wave loading in a probable maximum flood (PMF) event. If the wave wall were to fail in a PMF event, the effective dam wave freeboard would be marginal. There was insufficient evidence to recommend replacement of the wave wall on reservoir safety grounds, but works would be required to remediate the existing wave wall, including a section of the wall adjacent to the spillway bridge, which had rotated, possibly indicating a loss of support due to erosion. The existing spillway bridge was in poor condition and impinges on flows over the spillway weir in a PMF event.

To address the findings of the study, the scope of work required was:

1) Construction of a reinforced concrete watertight spillway lining to fully contain design flows, including localized wall heightening,
2) Removal of the existing baffle blocks and guide wall,
3) The addition of shear keys for spillway chute stability,
4) Grouting works to close voids and to enhance the existing grout curtain,
5) Addition back of wall drainage along the right side of the spillway chute,
6) Earthworks to achieve the required finished ground profile adjacent to the spillway walls, and
7) Remedial/repair works to the existing wave wall and spillway approach the wall~~,~~ [30].

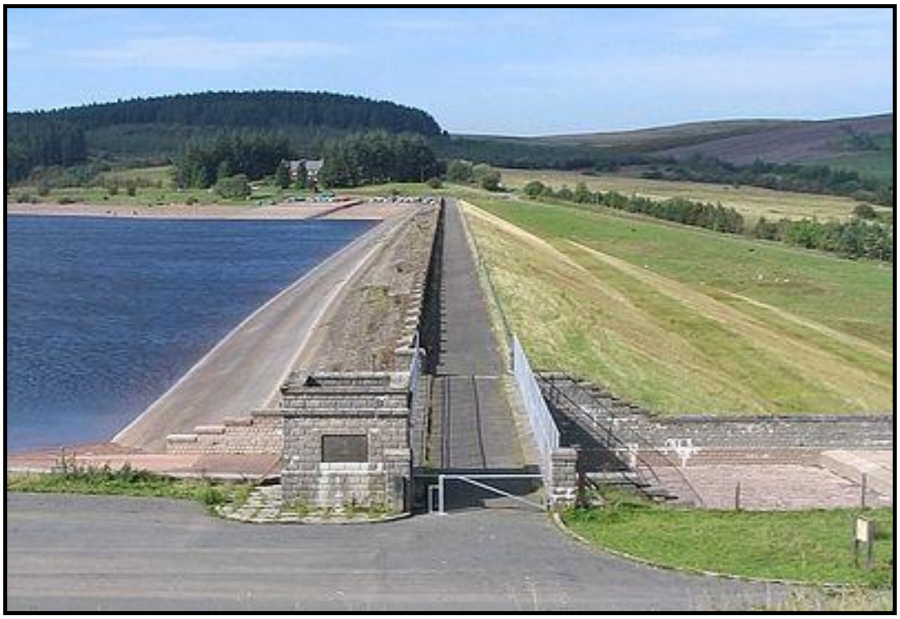


Figure 20: Usk Dam Reservoir [31].

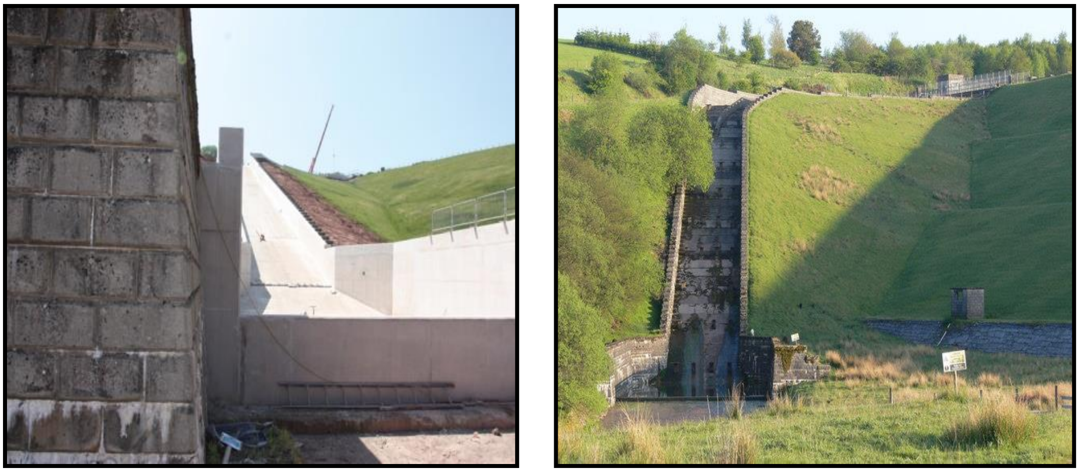


Figure 21: Usk Reservoir Dam Spillway views from downstream, after renovation (Left), before renovation (Right) (Photo by Mat Fascione) [32].

7.8 Folsom Dam- USA

Folsom Dam is a concrete gravity dam on the American River of Northern California in the United States. It was completed in 1955, officially opened the following year. The concrete dam is 100m high and 430m long, flanked by earthen wing dams, refer Figure 22 [33].

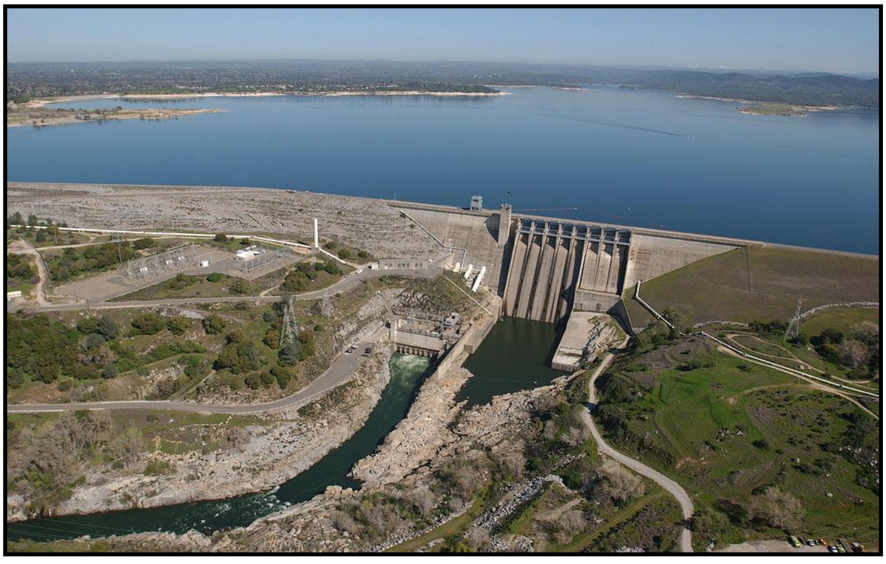


Figure 22: Folsom Dam General View [33].

The reservoir is held in place by additional nine saddle dams in the west and southeast sides. The wing dams total a length of 2,700m, while the saddle dams measure 5,040m long combined. The dam and appurtenant dikes total a length of 8,150m. Floodwaters are released by a spillway located on the main dam, controlled by eight radial gates with a capacity totaling 16,100m^3/s as well as a set of works with a capacity 3,300m^3/s. The dam and its reservoir, Folsom Lake, are part of the Central Valley Project, a multipurpose project that provides flood control, hydroelectricity and irrigation and municipal water supply. In order to increase Sacramento's flood protection to 200 year, flood protection (meaning that the area is protected from a flood that has a 0.5% chance of occurring in any given year) the US Army Corps of Engineers recently constructed an auxiliary spillway, which was completed in October 2017 and enables Folsom Dam operators to increase outflows to prevent the lake level from reaching or exceeding the height of the main dam gates [34].

The volume of impounded water behind the dam forms Folsom Lake at the normal pool level is 1.201km^3 and a surcharge capacity of 0.14km^3 making a total capacity of 1.341km^3. The original capacity was 1.25 km^3, but it has been reduced somewhat due to sedimentation. The dam and reservoir control runoff from a catchment area of 4,860km^2, or 87.6% of the 5,500km^2 American River watershed. The average amount of runoff entering the reservoir is 3.3km^3, forcing the release of 2.1km^3 for flood control. During a severe storm in December 1964, the inflow into Folsom Lake reached a record high of 7,900 m^3 per second, with a river release of 3,300m^3 per second. In February 1986, nearly 500,000 people faced the possibility of flooding when engineers at Folsom Dam were forced to open the spillway gates after heavy rains. The flooding was made worse by the failure of the Auburn Dam (a cofferdam) upstream which released an extra 120,000,000m^3 into the American River. A peak flow of 7,100m^3/s entered Folsom Lake forcing operators at Folsom Dam to open all the spillway gates releasing 3,700m^3/s into the American River. This was 420m^3/s more than the safe capacity of downstream levees. Although the dam and the Sacramento levee system held without major damages, the requisite winter flood control space was increased later on by 50%, from 493Mm^3 to 740Mm^3 to protect against future floods. In addition, about 41,000,000m^3 of sediment carried down from the mountains were deposited in Folsom Lake, considerably reducing its capacity. The consequence was a reduced capacity to store winter rainfall for summer use. The New Year's Day storm of 1997 was the most severe in recent history, with a total inflow of 1,223Mm^3 (equal to the entire capacity of Folsom Lake) over a 5-day period. However, this time the Bureau of Reclamation was able to limit releases to less than 3,100m^3/s.

The 1997 storm was a classic example of a "rain on snow" event, during which a warm tropical storm melted existing snowpack at lower and middle elevations, effectively doubling the volume of runoff. Prior to the New Year's storm, the winter of December 1996 had also been one of the wettest ever recorded, saturating the ground and depositing a considerable amount of snow [35]. The Bureau of Reclamation's Safety of Dams Program determined the risk of flooding in the Sacramento area made it one of the most at-risk communities in the United States. Two projects to increase flood protection have been undertaken. The first will raise the surrounding dikes by 2.1m to increase flood protection. The second, a new additional auxiliary spillway is designed to handle the runoff from large storms and snowmelt floods that might cause damage in the region. The new spillway is built with gates 15m lower than the existing spillway, allowing for more efficient evacuation of reservoir storage before flooding events. The auxiliary spillway was constructed adjacent to Folsom main concrete dam, and it was substantially completed in 2017. It includes a 1,100-foot-long approach channel beginning in Folsom Reservoir, a concrete control structure with six bulkhead and six radial gates, a 3,100-foot-long auxiliary spillway chute, and a stilling basin that acts as an energy dissipation structure as water discharges and enters the American River, refer Figure 23 (Left) [35] and Figure 23 (Right) [36].

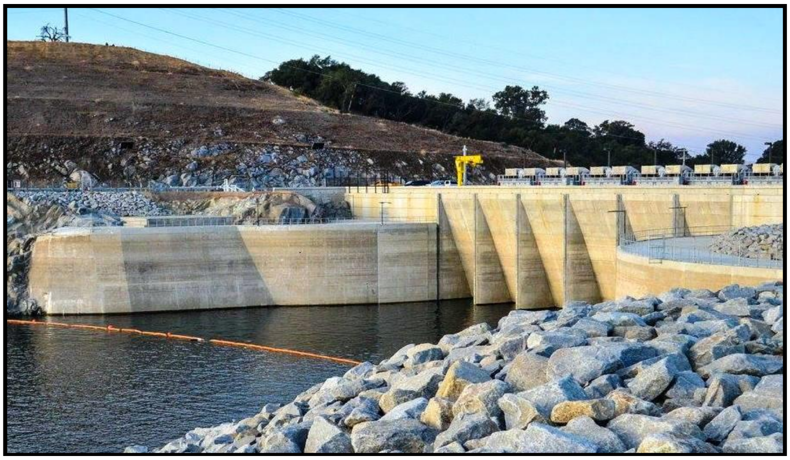


**Figure 23: (Left) Auxiliary Spillway before completion [35].
(Right) Auxiliary Spillway after completion [36].**

The spillway was substantially completed in October 2017, refer Figure 24.

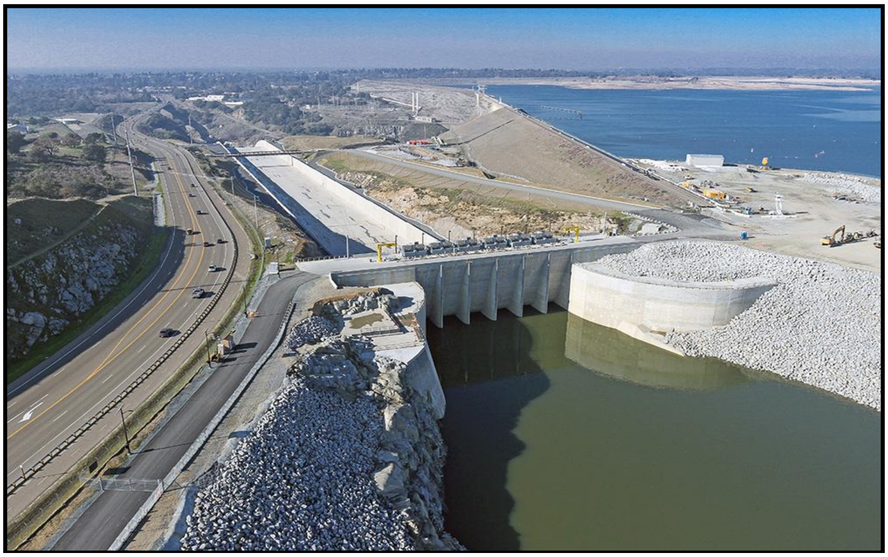


Figure 24: View of Folsom Dam Auxiliary Spillway after completion showing head works and chute [37].

With the ability to operate the new spillway, the USBR is able to better manage large floods by safely releasing more water from Folsom Reservoir earlier in a storm through both the spillway gates on Folsom Dam and the new control structure's radial gates (which sit 15m lower), reducing hydrologic risk and leaving more storage capacity in the reservoir.

In Summary, of overtopping, it may be said that enhancing dams' safety against overtopping may include various actions involving modifications to the spillway or the dam or both. With respect to spillways, once the reviewed design flood is found to be significantly higher than the original design flood, adequacy of spillway capacity needs to be thoroughly reviewed. Solutions may vary between, 1) augmenting the existing spillway capacity through addition of more spillway bays of the same type as existing, 2) provision of breaching sections or fuse plugs; if suitable sites are available. It is preferable to locate such breaching sections on a saddle rather than on the main dam section, or 3) adding a completely new auxiliary spillway.

In all the above cases; however, it is required to investigate the alignment of the surplus water channel till it meets the main channel to assess the likely damages in the surrounding valley in the event of design flood causing a breach, and to make sure, also, that increasing the maximum design outflow from the dam appurtenant structures did not lead to increasing the flood hazards risks to the downstream communities.

Other actions involving the dam itself may be sought by adopting one of the following alternatives depending on their suitability with respect to site conditions, these may be such as: 1) Increasing the freeboard above the maximum designed retention level by a provision of parapet, including strengthening of sections; where necessary so that the flood cushion available will be increased, 2) there may be cases where the dam height itself may have to be increased if freeboard is not sufficient to allow temporary raising of the reservoir level. In such case, structural stability analysis will be made to ensure the stability of the dam against the increased water load, and the safety of the structure must be confirmed.

In cases where structural modifications to the dam or the spillway are not possible then it may be necessary to change the reservoir operation rules to ensure the safe routing of the revised inflow design discharge. In such case, this can be done by increasing the flood storage by lowering the conservation storage level, so that flood moderation will be enhanced, and this may involve in some cases lowering the spillway sill level. This may result; however, in reduction of the benefits such as reducing power generation and/or irrigation storage, but it adds on the positive side as it will involve little investment required for modification; if any.

8. Climate Change and Dams Safety

The operation and planning of basin-scale water resources systems have historically relied on assuming stationary hydrologic conditions. This approach assumes that past hydrologic conditions are sufficient to guide the future operation and planning of water resources systems and infrastructure. This assumption is threatened nowadays by climate change, and to the notion that both climate and hydrology will evolve in the future [38].

Global climate change is expected to lead to changes in climatic factors, such as, variations in extreme temperatures and frequency and extent of heavy precipitation events, which may produce conditions outside current design criteria for dams in many parts of the world.

Changing precipitation temporal patterns together with rising temperature, as may be expected in many cases from climate change, can aggravate flow conditions leading to unprecedented floods. For example, in regions where seasonal snowmelt plays a significant role in annual runoff, hotter temperatures can trigger more rain-on-snow events, which induce faster and often earlier melting such that the combination of rain and melting snow can aggravate spring flooding in a short period of time. Moreover, during the early spring, frozen land prevents melting snow or rainfall from seeping into the ground leading to larger volumes of surface runoff. If seasonal storms are added to all this result is often severe spring flooding. Spring`s storms intensified by climate change factors can also bring several inches of precipitation in just few hours. These heavy rains can lead to severe flooding by over saturating the ground and may lead to earlier filling of reservoirs and leaving no room for next flood events and present chances for dams overtopping. U.S.A., as an example, has thousands of miles of levees and dams that are designed to protect against a certain level of flooding. These structures can erode and weaken over time, and they can also be overtopped, or even fail, during larger flood events [39]. According to the United States Association of Dam Safety Officials, 73 dams have failed since 2010, not including the recent ones in South Carolina. The data shows that most of the failures were caused by extreme weather. The average age of the dams that failed was 62 years [40].

Large dams and other water resources' systems, as already mentioned have always been based on the assumption that future stream-flow patterns will mirror those of the past, but this is no longer true. Climate change has begun to significantly and unpredictably change precipitation patterns.

In looking to the full picture of climate change on the World scale, more frequent droughts will make many hydropower projects uneconomic, while more extreme rainfall will increase siltation of dams (reducing their useful lifetimes) and increase the risk of dam overtopping failures and catastrophic flood releases.

Unfortunately, large dams' developers do not currently take climate change into account in their plans. If they do, dams would need much greater capacities to safely pass high floods, and projections of power generation for hydropower projects would have to allow for the probability of new extremes of drought. These factors would increase the costs and reduce the benefits from dams, thus making other alternatives available to them more attractive.

Predictability of climate change impacts on any particular area or region, however, is not an easy task and may require many detailed studies. This has put a special duty on the various governments to draw up plans regarding climate change impacts on the various aspects of life, which include among other things dam safety. Examples to this are the studies performed by the Scandinavian countries, Norway and Sweden, to face these challenges:

The first is the study of the Norwegian Ministry of Environment on adapting to changing climate issued in 2010. In this report, forecasts on climate change impacts on climatic factors are forecasted, and hence their effects on all aspects of the socio-economic issues in Norway are predicted. A comprehensive action plan with recommended future works is then given to mitigate these impacts. With respect to flood protection and dams' safety, it stipulates the following:

"*Since dam bursts (Overtopping) may pose a risk to human life, dam safety is given special attention. The regulations for dam safety comprise all dams that can potentially injure a third party. There are approximately 1700 hydropower dams in Norway and around 750 of these dams represent a risk to human life if they should burst. The regulations for dam safety stipulate that dams must be assessed every 15 to 20 years, depending on their consequence class. The purpose of this reassessment is to identify any changes in the loads, such as the size of floods, wind waves, ice pressure, etc., and changes in the regulatory requirements. The design flood magnitude should, for example, be recalculated so that any changes over the past years are identified, including changes as a result of climate change. It appears that the requirements and expectations that are stipulated in statutes and regulations provide satisfactory legal authority for continuous climate change adaptation in the sector, and to ensure that future changes in the climate are taken into account"*.

It goes on to recommend the following when applying for a license to build a new dam;

"Climate effects should be a key topic in the impact assessments that are required in connection with the license applications", [41].

In Sweden, the issue of climate change impacts on dam safety is also taken up very seriously. In a statement published by the Swedish Portal for Climate Change Adaptation, which is the result of cooperation between nineteen Swedish Government Agencies, the following is stated under the title of "Dam safety-Sweden" last updated on April 28, 2016.

"*Changes in high water flows are the dominant influence on climate change and dam safety. Adaptation measures to climate change for hydroelectric dams include reconstruction measures as well as operating and water conservation measures"*.

It goes on to state:

"*If the inflow in a particular situation exceeds the dam plant's releasing capacity the water level will rise in the reservoir. If the reservoir rises above the* *dam's limit, exceeding its dimensions, it could lead to dam failure. Adaptation measures for hydroelectric dams include reconstruction measures as well as operating and water conservation measures. Other climate indicators such as wind, torrential rain, temperature changes, ground frost and snow affect dam safety, to a certain extent, but not as much as extreme flows"* [42].

9. Conclusions

Thousands of dams have been built in the world during the last one hundred or more years. But most of the older dams were planned and constructed using very little hydrological data and utilizing primitive hydrological methods to estimate the inflow design discharge. This led to many catastrophic overtopping failures causing a great number of fatalities and extensive material damages. The question of safety against overtopping stems from two factors: mainly spillway adequacy which is derived from the fact that new methods of analysis and new hydrometeorological data may indicate the use of a larger spillway design flood than the one used in the original design, and also not having enough freeboard. An extensive revision of the safety of a great number of dams is therefore warranted nowadays, especially that advanced tools of such revisions are available, which include calculation procedures, such as the accurate estimation of the Probable Maximum Floods, and advanced statistical tools to analyze large volumes of flow data and meteorological information, which have been obtained by refined measuring methods. In this case, the probability of exceedance of any value can be reasonably found which is compatible with the require safety level. Upgrading of dams should follow this step to bring these dams to an acceptable standard of safety. The other source of urgency for revisions of dams' safety against overtopping, among other failure modes, stems from the new and strict government authorities' regulations requiring risk reduction towards communities that live downstream from dams. It is good to see now that large efforts are being made for upgrading dams, for which only few examples are given in the proceeding paragraphs. Notwithstanding all these efforts; however, the new awareness of climate change impacts on the hydrological cycle should be another reason for governments to draw up plans and issue regulations to consider the risks involved and take necessary actions to mitigate the impacts of climate change on the hydrological cycle, whether in reviewing already constructed dams or licensing the construction of new ones.

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Dam Safety and Earthquakes

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Abstract

Earthquakes may cause failure or profound damage for dams. Factors contributing to this are, magnitude on the Richter scale, peak horizontal and vertical accelerations, time duration, in addition to the epicentral distance, nature of foundation rock, criteria of the design, and finally, if appropriate type of dam and materials has been used. Extensive lists of dam failures and damaged once are presented with many case histories. Most failed dams were tailing dams or hydraulic fill dams or small earth fill dams, which reflect the weight of the design and construction factors. Embankment dams, normally, are less tolerant to ground shacking than concrete dams. While rockfill and RCC dams have shown good performance. The developments of design methods and criteria are traced here, from the early use of the pseudoptotic method to the more rational dynamic analysis, which is used nowadays making construction of very large safe dams in seismic regions possible. The method adopts peak ground accelerations from anticipated earthquakes as inputs to the analysis which produce a full spectrum of the factor of safety during any considered event. This has led to increased use of seismic instrumentation to produce seismographs of actual events in the free field, and on dams hit by earthquakes for comparison with outputs of this analysis and for future use for similar dams in similar circumstances, and to decide on rehabilitation measures. The safety levels to which any dam is to be designed are defined in terms of the Maximum Credible Earthquake, Safety Evaluation Earthquake, Maximum Design Earthquake and other similar terms. Dam repairs after sustaining earthquake damages are described in real cases and upgrading of older dams to withstand higher expected seismic events are also treated here and supported by case histories.

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1. Earthquakes in General

An earthquake is the shaking of the Earth surface resulting from a sudden release of energy in its crust and its uppermost mantle, which constitute its hard and rigid outer layer, and, therefore, creating elastic waves known as seismic waves. Earthquakes can range in size from those that are so weak that they cannot be felt to that violent enough to toss people around and destroy whole cities. The seismicity, or seismic activity, of an area are the frequency, type, and size of earthquakes experienced over a period of time. Earthquakes and the release of its energy are caused mostly by rupture of geological faults but also by other events such as volcanic activity; landslides, mine blast, and nuclear tests. An earthquake's point of initial rupture deep, down is called its focus or hypocenter, and the epicenter is the point at the ground level directly above the hypocenter. The mechanism of this rupture is attributed to the fact that the upper mantle of the Earth consist of seven or eight major tectonic plates, (depending on how they are defined), and many minor plates, which are in constant motion Figure 1 [1]. Where the plates meet, their relative motion determines the type of boundary: convergent, divergent and transform. Earthquakes, volcanic activity, mountain-building, and oceanic trench formation occur along these plate boundaries (or faults) which form the seismic belts of Earth. The two major seismic belts are the Circum-Pacific Belt, which surrounds the Pacific Ocean, and the Alpine Belt, which stretches from the Azores through the Mediterranean and Middle East to the Himalayas and Indonesia, where it joins the Circum- Pacific Belt. A purely oceanic seismic belt lies along the mid-Atlantic ridge, [2]. Along these belts the majority of earthquakes epicenters cluster, refer Figure 2 [3].

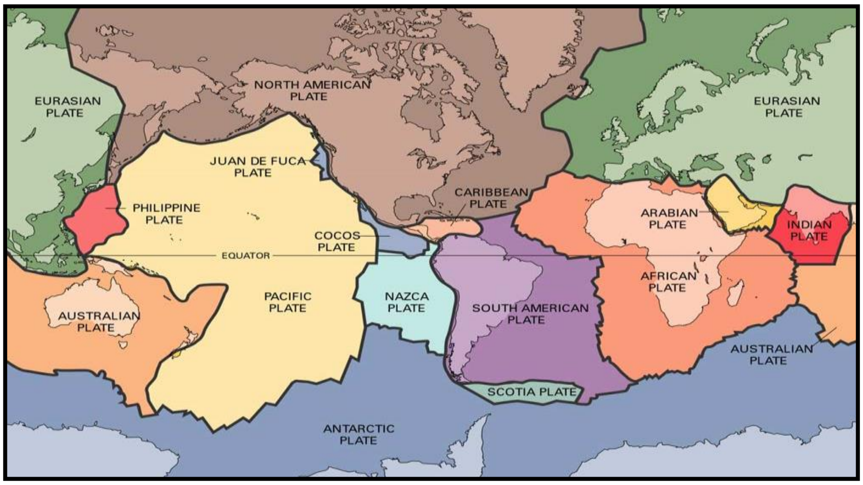
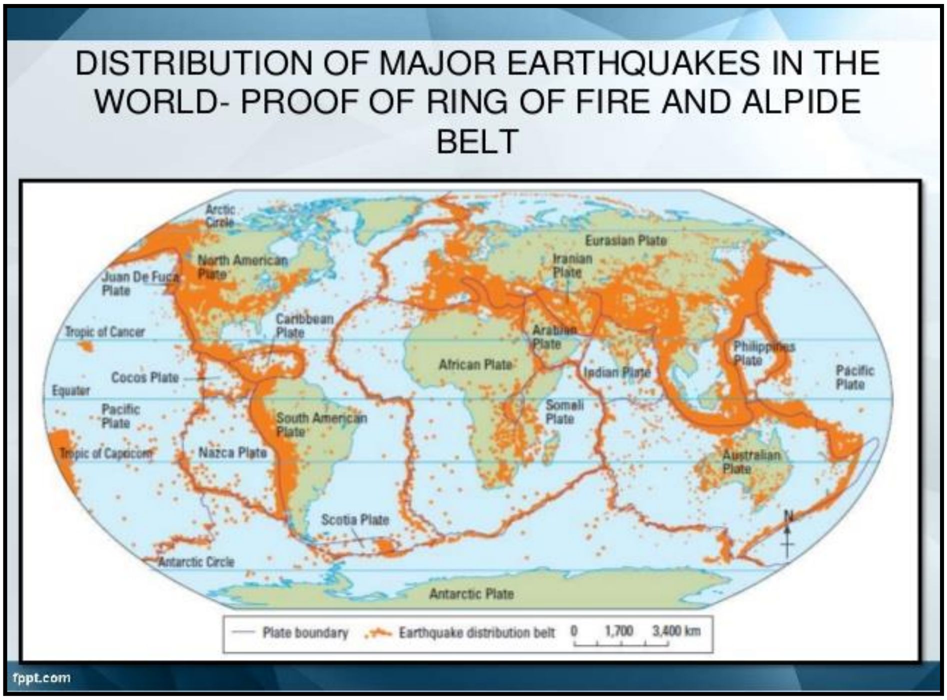


Figure 1: The Earth's tectonic plates. (Source: USGS) [1].



DISTRIBUTION OF MAJOR EARTHQUAKES IN THE WORLD- PROOF OF RING OF FIRE AND ALPIDE BELT

Figure 2: Distribution of Major Earthquakes in the World [3].

The location of any point on the earth surface close to any of these belts indicates high probability of earthquakes occurrence at that point. It is the magnitude of any earthquake, which represents the governing factor for the extent of damage that can happen to structures, whether being dams, buildings, bridges or any other infrastructure or lifeline. Local faults, even not related to these major seismic belts, can also give rise to earthquake and result in such damages and even loss of life. The obvious conclusion that may be drawn is that, it is the duty of the designing engineer to ensure the safety of these structures against the impacts of earthquakes by using building codes. In major structures such as dams, Atomic Energy Power Stations and the like, a complete Seismic Study is not only justified but strongly recommended.

2. History of Dams Failures and Incidents due to Earthquakes

During late history, not many dams have collapsed or very badly damaged worldwide by earthquakes. Most failed dams were tailing dams or hydraulic fill dams or small earth fill dams. Few embankments or concrete gravity dams of significant size have been severely damaged. This good record may be largely due to the fact that few dams have been shaken by earthquakes of duration and intensity sufficient to jeopardize their structural integrity. It is well understood also that the failed dams, especially tailing dams or the small earth dams were not designed or constructed to meet rigid criteria for safety under earthquake loading.

In Table 1 a list is given of collapsed dams that have experienced significant earthquake shaking from 1886 to 2000. It includes, where available, principal earthquake parameters, dimensions and types of dam, epicentral distances. Table 2 lists also dams which were severely damaged (but did not fail) during the same period [4]. Examining these two tables indicates that most of the failed or the seriously damaged dams were tailing dams and/or fill dams of generally low heights and very few large concrete and earth fill dams. The explanation to this is already given.

Apart from the mentioned dams (Tables 3 and 4), there were hundreds of other dams, which had sustained minor damages or not damaged at all which may be referred to in reference [4].

Referring back to these two tables it may be concluded that they infer the fact that dams behave differently in response to earthquake events depending on the strength of the earthquake on the Richter Scale (M), type of the dam and its location relative to the epicenter of the earthquake. Moreover, Table 2 does not give full description of damage and characteristics of the site and more details of the dams themselves. To explain these points some important earthquakes are given hereunder with their impacts on dams within their area of influence for clarification.

3. Important Earthquakes and their Observed Damages on Earth fill Dams

3.1 General

Studying previous major earthquakes and the damage they left on dams serve to illustrate the possible performance of similar dams in similar future seismic events. The lessons learned from such case histories help designers in selecting the type of dam best suited to the site under consideration, materials for construction and the seismic criteria to be followed in addition to other design details.

Table 1: List of reported dams which failed as a result of earthquake (1896- 2000)[4].

Dam Name	Country	Type	Height [ft]	Earthquake name	Earthquake Date	Magnitude	Distance [km]
Augusta	USA	E	-	Charleston	13 Aug 1886	7.0	180,0
Vulcano Lake	Mexico	E	12	Imperial Valley	22 June 1915	5.3	0.0
Fairmont	USA	E	-	Imperial Valley	22 October 1916	5.0	22.0
Sheffield- 2	USA	E	25	Santa Barbra	29 June 1925	6.3	11.2
Barahona	Chile	T	200	Talca	01 October 1928	8.4	160,0
Vulcano Lake	Mexico	E	12	El Centro	18 May 1940	7.1	0.0
Hosorogi	Japan	E	28	Fukui	28 January 1948	7.3	4.8
Coleman	USA	Comp	-	Fallon	23 August 1954	6.7	24.0
Saguspe	USA	E	-	Fallon	23 August 1954	6.7	24.0
Rogers	USA	M	-	Fallon	23 August 1954	6.7	80.0
El Soldado	Chile	T	-	Chile	28 March 1965	7.1	-
El Cobre	Chile	T	-	Chile	28 March 1965	7.1	35.0
Hayagakenuma	Japan	E	40	Tokachi- Oki	16 May1968	-	-
Ichrigoya	Japan	E	26	Tokachi- Oki	16 May 1968	-	-
Gamanosawa	Japan	E	34	Tokachi- Oki	16 May 1968	-	-
Shorey	Peru	T	-	Peru	1969	-	-
Huachopolca	Peru	T	-	Peru	1970	-	-
Salamanca	Chile	T		Chile	08 July 1971	7.5	110,0
Illapel	Chile	T	26	Chile	08 July 1971	7.5	100,0
Cerro Negro	Chile	T	-	Chile	08 July 1971	7.5	-
Mochinkoshi 1	Japan	T	98	NrIzu- Oshima	14 January 1978	7.0	35
Cerro Negro-2	Chile	T	105	Chile	03 March 1985	7.7	-
Veta De Aqua	Chile	T	-	Chile	03 March 1985	7.7	-
Upper Koyoen	Japan	E	30	Kobe	17 January 1995	6.9	-
Central Koyoen	Japan	E	30	Kobe	17 January 1995	6.9	-
Niteko	Japan	E	-	Kobe	17 January 1995	6.9	<10
Shih- Kang	Taiwan	CG	82	Chi- Chi	17 September 1999	7.6	0.0

Legend E= Earth dam. **Comp** = Composite (fill/ Concrete), **M**= Masonry, **T**= Tailing, **CG**= Concrete Gravity

Table 2: List of reported dams which were severely damaged (but did not fail) as a result of earthquakes [4].

Dam Name	Country	Type	Height [ft.]	Earthquake name	Earthquake Date	M	Distance [km]
Ono	Japan	E	161	Kanto	01 September 1923	8.2	51.0
Misc. Embankments	Japan	E	50/8	Ojka	1939	6.6	-
Hebgen [1]	USA	E	90	Hebgen Lake	17 August 1959	7.1	16.0
Hsinfengkiang	Chile	CGB	344	Hsinfengkiang	19 March 1962	6.1	1.1
Bella vista	Chile	T	-	Chile	28 March 1965	7.1	55.0
Koyna [1]	India	CG	338	Koyna	11 December 1967	6.5	3.0
Yeyuan	China	E	82	Bohai Gulf	18 July 1968	7.2	?
U. Van Norman	U SA	HF	80	San Fernando	09 February 1971	6.5	11.2
El Cobri	Chile	T	-	Chile	08 July 1971	7.5	80.0
Lliu	Chile	T	-	Chile	08 July 1971	7.5	-
Shimen Ling	China	E	147	Haicheng	04 February 1975	7.3	33.0
Touho (Douhe)	China	E	72	Tangshan	28 July 1976	7.8	-
Mochinkoshi No2 [2]	Japan	T	98	Nr i- O Atssshk	15 January 1978	5.8	-
La Palma	Chile	T	26	Chile	03 March 1985	7.7	-
Austrian [1]	USA	E	185	Loma Prieta	17 October 1989	7.1	
Masy way [2]	Luzon	E	82	Philippines	16 July 1990	7.7	19.2
Niwajkumine	Japan	E	?	Hokkaido Nans	12 July 1993	7.8	74
Lower San Fernando	USA	HF	125	Northridge	17 January 1994	6.7	9.4
Lower Koyoen	Japan	E	30	Kobe	17 January 1995	6.9	-
Zhong Hai	China	CG	82	Lijang	03 February	7.0	4.0

Legend E= Earthfill. **CGB**= Concrete Gravity Buttress. **CG**= Concrete Gravity. **HF**= Hydraulic Fill. **T**= Tailing

Major earthquakes, as natural phenomena, have in most cases left behind destruction and life losses, which have led to a considerable number of studies and detailed documentations. One of the important aspects so covered in these documentations is the damages earthquakes have left on dams that were located in their area of influence. It is of interest to note that ICOLD has devoted many of its publications to highlight seismic action on dams together with other national committees on large dams and government agencies.

From following case histories of dams damaged by earthquakes, but did not fail, many serious conclusions can be derived. To follow these case histories in more details one may refer to references [5] and [6], given in the list of references. In the following some of the important factors contributing to damage suffered by dams as a consequence of earthquakes, are outlined:

1. The magnitude of damage on dams is generally commensurate with the magnitude of the earthquake. As one example; the 1923 Kanto, Japan`s earthquake (M= 8.2) damaging the 122 feet high Ono earthfill dam. The dam

was fractured in many places with one vertical fissure extending 70 feet along the puddle clay core and settlement of about one foot in addition to longitudinal crack on the crest of ten inches width and length up to 200 feet. Similarly, the case of the 1985 Mexico earthquake (M=8.1) which damaged the 197 feet earth rockfill and central clay core La Villita dam, and the 485 feet high el Infiernillo rockfill and earth core dam. Both dams experienced considerable settlements and small permanent deformations. These damages were added to similar damages they both had received in the previous five events between 1975 to 1985, which had magnitudes exceeding 7.0, but without failing. This may be compared with the 1987 Whittier Narrows (M= 6.1) earthquake, which affected many embankment dams in greater Los Angeles area and showed no significant damages as indicated by the strong motion records obtained from this event.

2. The mode of ground shaking during the earthquake may contribute also to the damage produced on structures within the epicentral area, including dams. In the 17th of October 1989 Loma Prieta event, California, USA (M= 7.1), about dozen dams located within the epicentral area withstood the strong ground shaking. This was because the strong phase of shaking (acceleration > 0.05g) during the earthquake lasted less than eight seconds at rock and firm soil sites.
3. It was proved also from documenting various earthquakes that embankment dams' behavior during earthquakes in which sound seismic design criteria were used, was relatively better than other dams, as they sustained lesser damages. Good examples may be cited from the 17th September (M=7.4) Koçaeli earthquake in Turkey. This earthquake was caused by the rupture of the northernmost strand of the North Anatolia fault system, which produced seven earthquakes with magnitudes more than 7.0 since 1939. None of the 48 dams located within the area were affected. Two of the dams closest to the area where the recently completed, Yuvaçik Dam, which was about 7km of the earthquake's epicenter, and the Gokçe Dam, located around 55km to the southwest close to the town Yalova.

Yuvaçik a Dam, a 108m high rock and gravel earth fill and clay core dam was first impounded in June 1998. Its live storage capacity is 55Mm^3, and its full capacity is 66 Mm^3. Sound seismic design criteria were used for its design by considering horizontal peak ground acceleration of 0.15g for the dam, and with a lower acceleration used for design of the associated structures. The dam incurred very little damage as a result of this event. The fact that the reservoir was not full was fortunate as the freeboard allowed for wave action, and settlement was only 1.5m. Total settlement before the event measured only 25mm, and following the earthquake the maximum settlement was only 130mm, but it was reported, however, that the tidal wave induced during the event had a height of 2. m. The earthquake resulted in almost negligible horizontal movement of the dam of about 30mm, with the dam recovering to half of this value within weeks from the earthquake.

The Gokçe Dam is an embankment dam 50m high and similar to Yuvaçik Dam, its reservoir was only impounded to half of its full capacity of 25.5Mm^3 at the time of the earthquake and was designed according to the same seismic criteria also. This dam was found to have suffered no obvious damage during the event. The intake structure is similar to that on the larger Yuvaçik Dam, and, while it showed evidence of some cracking; it appeared generally to be very robust.

In another case, on January 7th, 1994 Northridge earthquake (M=6.7), California USA, the earthquake induced ground motions, which were quite severe at 105 dams located within 75km radius of its epicenter. These dams were mostly the same dams shaken in 1971 during San Fernando (M=6.6) earthquake. Eleven earth fill and rockfill dams experienced some cracking and slope movements as a result of the last earthquake, yet none of them presented an immediate threat to life and property. This satisfactory performance may be due to a large extent, from the fact that in California, most significant dams have been reevaluated for the Maximum Credible Earthquake (MCE), during investigations initiated after the San Fernando Earthquake in 1971. Questionable or unsafe embankments have been upgraded or decommissioned, or the owners have been asked to operate with partially full reservoirs with an increased freeboard. In this connection, one of the few embankment dams that suffered noticeable damage from the Northridge Earthquake was the 125-foot high Lower Van Norman Dam, a hydraulic fill dam. The dam had been abandoned as a water storage facility since 1971 San Fernando earthquake, but was still used with empty reservoir for flood control. It experienced two to three and a half inch-wide cracks of several hundred feet long. Some of these cracks were at least five feet deep. Sand boils and a sinkhole were also observed along the upstream face. Maximum crest settlement was eight inches, and maximum horizontal crest movement was about four inches toward upstream. The 82-foot-high Upper Van Norman Dam, which was also left with an empty reservoir since it was severely damaged in 1971 experienced transverse cracks near its right abutment, on the downstream slope, and near its left abutment, which were up to 60 feet long and two to three inches wide. Maximum non-recoverable crest displacements were about 2.4 feet of settlement and over six inches of horizontal upstream movement. It is worth mentioning that the seismicity of the area where the two reservoirs were located was studied by the US Geological Survey in 1974 following the San Fernando earthquake, and the expected earthquakes` motion parameters were obtained and included in a circular of this department [7]. In the same event the 130 foot-high Los Angeles Dam, which has replaced the two Van Norman dams, and located between these two floods controls dry embankments, experienced extensive, but not safety threatening cracking of its asphalt lining and settled 3.5 inches near its maximum section. Maximum horizontal crest movement was about 2.2 inches. Lastly, the Northridge Earthquake caused minor damage in the form of transverse cracks and settlement to Lower Franklin Dam (103 feet high), Santa Felicia Dam (213 feet high), Sycamore Canyon Dam (40 feet high), Schoolhouse Debris Basin Dam (38 feet high), Cogs well Dam (266 feet high), Porter Estate Dam (41 feet high), and Rubio Basin Dam (64 feet high). Adoption of sound and proper design seismic criteria had saved all these dams from failure.

4. Selection of materials of earthfill dams may also contribute greatly to their response to earthquakes. Many earthfill dams were affected by the February 1971 San Fernando earthquake (M= 6.6) which experienced (0.15g) or greater ground shaking. The Lower San Fernando dam, a 140 foot high hydraulic fill dam, performed poorly and was severally damaged. The upstream face and the crest slid into the reservoir leaving only 5 feet of freeboard against overtopping. Lower Van Norman dam and the Upper Van Norman dam (mentioned also above in relation to 1994 Northridge event) were seriously damaged by this earthquake and were taken out of service as permanent storage dams but remained to serve for flood protection dams as already explained. The first is the hydraulic fill dam, so it experienced a widespread liquefaction and major slope failures. Overtopping did not occur only because the reservoir water level was relatively low when the earthquake struck. The Upper Van Norman dam was also severely damaged.

These cases brought to the attention of engineers the potential vulnerability of embankments constructed of poorly graded, badly compacted and saturated fine sands and silt~~s~~, and led to significant advance in the numerical methods of dynamic analysis of dams. This also sheds light on the large number of tailing dams’ failures during earthquakes, which result as a consequence of the uniform graded silts or sand of which they are made of, in addition to the low degree of compaction and the high degree of saturation of these materials making them very susceptible to liquefaction.

In contrast to the above mentioned cases, the behavior of Kitamaya Dam in response to the January 17, 1995 Kobe, Japan (M=6.9) earthquake merits mentioning. This dam, an 80-foot high embankment dam was built of decomposed granite of varying gradations with a vertical chimney drain; it was about 31km away from the epicenter of this earthquake. At the time of the earthquake, the reservoir was at the maximum operational level. After the earthquake, reservoir drawdown revealed a 1.0 to 1.5m scarp and bulging at the toe, so test pits were excavated into the slope to determine the condition of the embankment at this location and to obtain samples for testing. Beneath the riprap and gravel bedding was a loose layer of rolled embankment, which was followed by an extremely loose layer that had contained the slide. Unaffected embankment was found below the sliding layer, and the sliding failure zone had a thickness of 1.5 to 2.0m. Undisturbed samples of the embankment above the sliding block were not particularly loose. The damages suffered by the dam were tolerable and repairable mainly because of good selection from the construction materials and degree of compaction. In the same event, no severe damages were observed in earth fill dams higher than 40 feet, which again was explained by the good selection of material and high degree of control during construction. Smaller embankment dams, however, suffered various forms of damages such as longitudinal cracking, transverse cracking, settlement, deformation of the dam body, and up to few complete failures; depending on their materials and construction practices exercised. In spite of the overall assessment of peak acceleration levels at these dams’ locations, which was estimated to be approximately 0.22g, the limited damages in this event could be partially explained by the fact that they had good rock foundations and give support to what has been mentioned in (3) above, in addition to a good selection of construction material, and good control over construction.

5. In many instances chance has contributed to save many dams, or at least, has helped to reduce damage in many dams. In these cases, the reservoirs of many of these dams were either empty or partially full during the earthquakes, so they were not subjected to the full hydrodynamic forces of water assumed for the design of the dam and its ancillary works such as gates [8].

This matter, taken with the other discussed factors, helped in clarifying the mechanism of dams' total or partial failures or even reducing the magnitude of the resulting damage. The example of Bhui (M=7.7), India, earthquake which struck the Gujarat Province on January 26, 2001, can serve as a good example. An important aspect of the Bhuj earthquake was the performance of about 300 small and moderate size embankment dams (according to ICOLD classification) that were constructed in this region in an effort to store water for irrigation and domestic water supply. All these dams were hit by a/m earthquake. Fortunately, at the time of the earthquake, being the end of the irrigation season, there was very little water in the reservoirs and most of the damage to embankments occurred in the valley section where the low pool kept the alluvium saturated. Following the event, the dams were categorized based on their observed damages, downstream consequences, and their importance to water supply. This classification was used to determine the appropriate course of action and prioritization for restoration. Damage classes and the total number of dams in each class are summarized in Table 3. Multiple post-earthquake investigators noted liquefaction related damage to embankments throughout the affected area, as evidenced by sand boils, ground cracking and lateral spreading. Many of these dams were constructed directly on loose alluvial deposits, and this was the reason behind the liquefaction of this material. But the fact that those dams were mostly empty contributed to the low hydrodynamic forces acting on them reducing damages [9].

The damages on these dams were of varying degrees of settlements, bulging of dam body, horizontal displacements, horizontal cracks at the top and upper zones of the dam of various depths and lengths, in addition to vertical cracks and slope slides, but no failures were reported.

Generally Speaking, very high dams, whether zoned earth fill or rockfill dams have responded fairly well during very strong earthquakes when they are designed according to sound seismic criteria using suitable zoned materials. Rockfill dams~~,~~ or concrete faced rockfill dams, however, showed in many cases superior performances over embankment dams of the same heights under the same conditions. The following examples are of severe events and their impacts on large modern designed, Earth fill and Rockfill dams.

Table 3: Damage classifications of earthen dams resulting from Bhuj 2001 earthquakes used to prioritize repairs.

Damage Class	**Description**	**Medium Dams**	**Minor Dams**	**Total Number**
1	Significant damage. Dam is critical to region and repairs will be completed before monsoon	4	7	11
2	Major damage. Not possible to repair before monsoon. Partial cut is made to allow water to pass, but allow for some storage	2	0	2
3	Major damage. Damage is too extensive to store water. Full cut is made to pass flood waters	1	3	4
4	Minor Damage. Repairs completed before monsoon	80	165	245
Total		**87**	**175**	**262**

The Wenchuan, China, May 12, 2008 (M=8) earthquake, which had struck the 512 foot high Zipingpu concrete faced rockfill dam. This Dam was 7km from the fault break, and it experienced an estimated peak ground acceleration of 0.5 to 0.6g. The crest settled 3 feet damaging small parts of the face slab. Maximum settlement was 760mm. The duration of strong ground motion was up to 120 seconds at sites underlain by deep alluvium.

In this earthquake, all the reservoirs in Sichuan and other seven nearby provinces were damaged. There were about 300 aftershocks some of which were over M=6. The number of earth fill dams which collapsed was 69 dam and there were 331 other highly dangerous situations. Well-built dams, especially rockfill dams, performed as designed [10], [11] and [12]. Nevertheless, the case of Bikou Dam, a 335 foot high earth fill dam with central core, has to be mentioned. It had experienced an estimated peak ground acceleration of 0.5g in this event, but its crest settled 9 inches only as a result of shaking, which is classified as mild damage [10].

Another very severe earthquake, the Mulue, Chile, February 27, 2010 (M=8.8) had its repercussions on large dams without any of them failing. One Example is Convento Viejo, 105 feet high, embankment dam, which was not damaged even though it experienced a peak ground acceleration of 0.38g, which was higher than what was expected when the dam was designed. At least 16 dams were moderately to severely be shaken with no reported failures. Some non-threatening slope failures, longitudinal and minor transverse cracking, however, occurred [13].

In the same earthquake, the Coihueco dam a 31 meter zoned earth fill dam suffered non-threatening sloughs on its upstream face and crest cracking apparently without liquefaction occurring [13] and [14].

The Tohoku, Japan, March 11, 2011 earthquake (M=9.0), when more than 400 dams, which were inspected afterwards, generally performed well with minor or moderate cracking occurring at embankment dams.

The Surikawa, a 172 foot high central core rockfill dam reacted well to this severe event. The dam settled a maximum of 7 inches; transverse cracking of the crest paving occurred near the abutments and the leakage temporarily increased from 18 to 25 gallons/minute. The measured peak horizontal accelerations were 0.11g at the foundation and 0.47g at the crest. The cracks were trenched and found to be only one foot deep.

The second high dam which was subjected to the same earthquake is the Kejauma, which is 79 feet high, central core rockfill dam. This dam experienced peak horizontal accelerations of 0.27 and 0.5g which was recorded at its foundation and crest, respectively. A maximum crest settlement of 6 inches occurred and leakage temporarily increased from 5 to 110 gallons/minute, and transverse cracking of the crest paving were up to one inch wide. A third dam, the Minamikawa Saddle Dam, a 64 foot high asphalt faced rockfill dam. The earthquake caused a temporary increase in leakage from 5 to 23 gallons/minute, a crack in the asphalt face, and a maximum crest settlement of 4 inches. A peak horizontal acceleration of 1.3g was measured at the crest. And, 0.27g was measured on the foundation of the main dam, 1km away. The analysis of accelerations, settlements and leakage increases measurements at these, and the other dams indicated the effects of the long duration of the M_w 9 earthquakes shaking.

An exceptional case, however, maybe that of the Fujinuma Ike dam, a 60 foot high, 436 feet long embankment where records show an inferior performance than the other dams. A preliminary report mentioned flaws in the embankment, such as thick lifts, so the dam may not have been constructed to modern standards. The report confirms the observation of long duration.

4. Important Earthquakes and their Observed Damages on Concrete Dams

Case histories have shown that concrete dams suffered less damage from earthquakes than embankment dams; this may be due to the nature of these structures, the type of construction material, degree of control exercised during construction, and to the relatively stronger foundations on which concrete dams are normally constructed as compared to their equals of earthfill dams. Apart from Shih-Kang dam, which failed on the 21st of September 1999, Chi- Chi Earthquake (M= 7.6) in Taiwan, no more concrete dams have failed, and this was the first case in history so far [15], [16].

It can be said that the performance of all types of concrete dams has been satisfactory. The Shih-Kang gravity dam experience, however, confirmed that concrete dams are vulnerable to major fault rupture.

Concrete buttresses dams when subjected to severe shaking have developed horizontal cracks at the elevation high in dams where the downstream buttresses intersect the vertical “chimney” section. This is an area where the stiffness of the concrete structures significantly changes.

Major thin arch concrete dam, with a full reservoir, although have performed in a good way, their behavior under peak ground acceleration exceeding 0.5g has yet to be known.

Some other specific conclusions on concrete dams where damage has been identified to indicate; there has been cracking high in the dam and where additional features such as curbs, railings, gates, or guard and/ or control houses are located. Cracking in buttress dams appeared to be due to upstream to downstream motions and not cross-canyon motions. Very little in the way of increased leakage has occurred in concrete dams subjected to major earthquakes. This can be explained, in part, to the fact that any cracking caused by the earthquake has mainly been horizontal and located high in the dam while the reservoir not being full in many cases. Some rock foundations have experienced a temporary increase in seepage following an earthquake which decreased later on.

There may be number of reasons why concrete dams have performed well and invariably better than that predicted.

The main reasons being:

1. Concrete dams are redundant structures that provide considerable capacity to redistribute load once damage occurs in the structure. Being so massive, typically there is plenty of concrete volume around damaged areas of the dam to carry loads around the damaged sections of the dam.
2. The duration of strong shaking may be too short to cause failure. Normally, it takes considerable time at high levels of shaking to cause failure of a medium-thick arch dam compared to a thin arch dam.
3. The dynamic tensile strength of concrete is taken as 50 percent higher than the static tensile strength of concrete. This increase in strength makes dams stronger during seismic shaking and increases resiliency.
4. Damping mechanisms can increase in the dam during the earthquake and reduce the seismic impact on the body of the dam. Damping increases still as the concrete cracks and contraction joints open and close.
5. The seismic impact of earthquake on the dam may be reduced because the natural frequency of the dam may not match the postulated frequency content of the earthquake taken for the design. For example, a gravity dam with a natural frequency of 7 Hz would not align with a ground motion with a peak spectral acceleration at 3 Hz.
6. The three-dimensional effects of the dam help prevent failure. The curvature in plain view of the dam or the narrowness of the canyon greatly increases the

seismic stability of a dam, and the potential for sliding of a gravity dam wedged in a narrow canyon is remote.

The generally accepted potential failure modes for concrete dams during an earthquake are cracking of concrete through the dam that forms removable blocks and sliding of the blocks during or after the earthquake. Severely shaken concrete dams; to date have cracked at locations of change in geometry (re-entrant corners) but have not formed removable concrete blocks. Thus, the entire potential seismic failure mode has not been fully achieved or experienced for concrete dams. While concrete dams are designed to withstand a higher degree of seismic shaking than buildings and have performed well in the past, we should not become overconfident of their performance in the future. Great care should be taken in the design details and quality of construction. Particular attention should be given to possible faults located directly under the dam.

In Shih-Kang gravity dam case, the dam was hit by Chi- Chi earthquake, which was caused from the Chelungpu faulting system rupture. Several concrete gravity and arch dams in the same area were severely shaken during the earthquake but performed satisfactorily. Shih- Kang dam itself could not withstand the earthquake force along the line of the fault crossed the dam itself causing considerable horizontal and vertical displacement, which acutely twisted the body of the dam causing it to fail. This high gravity dam is essentially a 18-bay gated spillway. The fault rupture extended both upstream and downstream of the dam and caused extensive damage to bays 16 to 18 on the right side of the structure [17]. As a general conclusion, it may be said that the performance of concrete dams has been satisfactory [18].

Perhaps hundreds or more concrete dams of all types had been shaken by earthquakes close to the dam sites, but only about 20 had experienced recorded or estimated peak ground accelerations (PGA)s of 0.2g or higher. Up to 2013, some of these dams have experienced Peak Ground Accelerations over 0.3g. The duration of motion of the M=9.0 Tohoku Earthquake was extraordinary long lasting from 150 to 300 seconds. Following the earthquake, about 240 concrete dams were inspected, and reports indicated that concrete dams appear to have performed very well during the main earthquake and numerous large aftershocks. One publication in 2017 gives a summary of concrete large dams that were shaken by a peak horizontal ground acceleration of more than 0.3g, Table 4. Peak accelerations at the crest were greater with full reservoirs, as expected. According to type, they were as follows:

Table 4: Number of Concrete Dams subjected to PHGA> 0.39 g showing type and corresponding damage [18].

	Dam Type	Number	Damage to dam	No Damage to dam	Minor Damage to dam
A	Gravity	10 (1 RCC)	1	5	4
B	Arch	6 (1 RCC)	1	2	3
C	Buttress	2	2	0	0
D	Spillway	1	1	0	0
Total		19	5	7	7

To demonstrate the relatively satisfactory performance of concrete dams, few reprehensive cases of all types are briefly described below to illustrate this point.

Examples of Concrete Gravity Dams:

1. Lower Crystal Springs Dam, this dam, a 127 foot high curved gravity dam hit by the San Francisco 1906 (M=7.9) Earthquake, but suffered no damage, although it was located only 0.25 miles from the San Andreas Fault. This dam also was been moderately shaken by the 1989 Loma Prieta (M=7.1) event without suffering damage.
2. Koyna Dam, India, this dam, a 338 foot high straight gravity dam was shaken by the 1967 Koynanagar (M=6.7) earthquake, which occurred near Koynanagar town close to dam site. The earthquake claimed at least 177 lives and injured over 2,200. The dam itself developed substantial longitudinal cracking at the top. Damage was attributed to design or construction details that would be avoided in modern structures [19].
3. Takou Dam, Japan, this is a (77m) 252 feet high straight gravity dam completed in 2006 which underwent the Tohoku 2011 (M=9) event and the following week (M=7.1) aftershock. In spite of the peak horizontal ground acceleration during the main shock which was estimated at 0.40g, the dam showed no damage except of an offset in the dam parapet wall and cracking of the wall of the gate house. This multipurpose dam had about half full reservoir during the main shock [20].

Examples of Concrete Arch Dams:

1. Gibraltar Dam, USA, this is a constant radius concrete arch dam 194.5 feet (59.3m) high and 600 feet (180m) long. It was severely shaken by the 1925 Santa Barbara Earthquake (M=6.3). The estimated Peak Horizontal Ground Acceleration (PHGA) was a greater than 0.30g. The dam; however, suffered no damage. The dam was strengthened in 1990 by additional supporting RCC weight on the dam downstream face, which, in effect, changed the dam from concrete arch dam to a curved gravity dam [18].
2. Pacoima Dam, USA, this is a 372 foot high concrete arch dam. It was shaken by the 1971 San Fernando Earthquake (M=6.6) and again in 1994 Northridge Earthquake (M=6.7). In the first event, a peak horizontal ground acceleration

of 1.25g was recorded on rock at left abutment, slightly above the dam crest. The depth of the reservoir at the time of this event was 60% of its impounding depth. It did not develop structural cracks or relative movements between adjacent blocks, but the left abutment had to be strengthened using post tensioned tendons to stabilize two large rock wedges that moved several inches as a result of the earthquake. In the 1994 event, the dam experienced ground acceleration also well above 1.0 g near the top of the same abutment. During this event, the reservoir was about one-third of its impounding depth. The dam suffered minor damages including movement of the joint between the left abutment block by 0.5 inch, and the opening of the left end of the dam by about two inches, while one location at the left abutment was displaced horizontally by 19 inches and a rock mass was displaced by 14 vertically in another location.

3. Rapel Dam, Chile, this is a double curvature arch dam 364 feet high and 886 feet long that was hit by two earthquakes. The first was the 1968 Santiago earthquake (M=7.8), and the second was the 2010 Maule (M=8.8) earthquake. Prior to the first event, a swarm of 300 earthquakes of lesser magnitude occurred. Measured peak free-field accelerations near the dam were 0.31g in the cross- canyon direction, 0.14g in the upstream to downstream direction, and 0.11g vertical. The arch dam did not experience any damage, but the appurtenant structures did have some damages. The spillway walls were cracked and there was leakage through the wall of the right spillway. The upper part of one intake tower cracked and separated from the dam. In the 2010 Maule earthquake, the reservoir was full, and measurements showed that the peak horizontal acceleration at the site was 0.3g. One concrete block of the dam at the left abutment of the dam which was next to the fault “Nido de Aquila” showed a rise of 0.02 inches. Seepage again increased along the right abutment; this time from normal 3.4gal/sec to 10.6gal/sec, and some concrete pavement at the dam crest cracked.

Examples of Concrete Buttress Dams:

1. Hsinfengkiang Dam, China, this is a 344 foot high buttress dam, which was shaken by the nearby earthquake of (M=6.1) in 1962. The dam developed substantial longitudinal cracking near the top, but damage was attributed to design and construction details that were avoided in modern structures.
2. Sefid Rud Dam, Iran, this is a 388 foot high buttress dam which was hit by the 1999 Manjil Earthquakes (M=7.4). It suffered severe cracking in the upper part of some buttresses and other forms of damages. It was rehabilitated and remained in services. Some 20 years later, a blister on the steel lining in the elbow of one of the two morning glory spillways in the left abutment appeared, and the increased leakage led into the discovery of previously unnoticed damage caused by the previous earthquake [17].

Examples of Rolled Compacted Concrete (RCC) Dams:

1. Shapai Dam, China. This is a 132m high RCC arch dam which was moderately shaken by the Wenchuan (M=8) in 2008. The dam withstood this event without damage to its body.
2. Miyatoko Dam, Japan. This is a 157 foot high RCC dam which was hit by the Tohoku (M= 9.0) earthquake of 11 March, 2011. The dam was located approximately 13km north of Sendai in Miyagi~~,~~ area where the peak horizontal ground acceleration was greater than 0.7g. A strong motion instrument located in the gallery recorded peak horizontal ground acceleration of 0.32g during this earthquake. No damage was recorded for Miyatoko and Takou Dams, and possibly others in Japan, will need to be confirmed at a later date [17].

5. Seismic Design Criteria for Large Dams

Large concrete dams were among the first structures for which seismic analysis and design had been performed. The seismic analysis method that was originally developed by Westergaard in the 1930s for the Hoover Dam has found worldwide acceptance among designers of concrete dams (Westergaard, 1933) [21]. This relatively simple pseudo-static analysis method accounts for both the inertial effects of the dam body and the hydrodynamic pressure acting on the vertical upstream face of a dam. It was a common practice to use a seismic coefficient of 0.1, corresponding to a horizontal force equals to the weight of the dam times a ground acceleration of 0.1g. The USBR, depending upon the size of the dam and the seismic risk, however, used seismic force which was assumed to range from 0.05 to 0.15 times the weight of the structure. For larger dams, the bureau combined horizontal acceleration effects with a vertical component, which was 50 percent of the horizontal acceleration; the assumed directions of the two components were those most unfavorable to structural stability. Most large dams design in countries other than USA adopted similar criteria. For example, Bhakra Dam in India, a 740-foot-high concrete gravity structure located about 180km from the epicenter of the Richter magnitude (M=8.6) Kangra earthquake of 1905, was designed for a lateral force coefficient of 0.15 and a vertical force coefficient of 0.075. The U.S. Army Corps of Engineers practice required the use of seismic coefficients for sliding and stability analyses of concrete dams and structures. Hydrodynamic pressures also were considered by similar methods in some cases [22]. The pseudo-static analysis method, however, proved in many cases to be unsatisfactory as it did not meet many seismic modes of ground shaking and ground acceleration. This method has therefore been replaced by the Dynamic Response Analysis, especially for large dams. Today, the seismic safety of dams is assessed based on typical failure modes and the inelastic deformations of dams due to ground shaking, and the evaluation of earthquakes. In this approach, the seismic design criteria for large dams define~~s~~ many levels of safety, as per the magnitude of the earthquake obtained from the seismic history of the area, and depending on the importance of the dam or related structures, as follows:

Maximum Credible Earthquake (MCE): It is the event which produces the largest ground motion expected at the dam site on the basis of the seismic history and the seismo-tectonic setup in the region. It is estimated based on deterministic earthquake scenarios. According to ICOLD, the ground motion parameters of the MCE shall be taken as the 84 percentiles (mean + one standard deviation).

Maximum Design Earthquake (MDE): For large dams, the return period of the (MDE) is taken as 10,000 years. For dams with small and/ or limited damage potential, shorter return periods can be specified. The (MDE) ground motion parameters are estimated based on a probabilistic seismic hazard analysis (PSHA). According to ICOLD, the mean values of the ground motion parameters of the (MDE) shall be taken. In the case where a single seismic source (fault) contributes mainly to the seismic hazard, uniform hazard spectra can be used for the seismic design. Otherwise, based on the disaggregation of the seismic hazard (magnitude versus focal distance) different scenario earthquakes may be defined.

Safety Evaluation Earthquake (SEE): The SEE is the earthquake ground motion which a dam must be able to resist without uncontrolled release of the reservoir. For major dams, the SEE can be taken either as the (MCE) or (MDE) ground motions. Usually, the most unfavorable ground motion parameters have to be taken. If it is not possible to make a realistic assessment of the (MCE); then the (SEE) shall be at least equal to the (MDE). The (SEE) is the governing earthquake ground motion for the safety assessment and seismic design of the dam and safety-relevant components, which have to be functioning after the (SEE).

Design Basis Earthquake (DBE): The (DBE) with a return period of 475 years is the reference design earthquake for the appurtenant structures. The (DBE) ground motion parameters are estimated based on a (PSHA). The mean values of the ground motion parameters of the (DBE) can be taken. Noting that the return period of the (DBE) may be determined in accordance with the earthquake codes and regulations for buildings and bridges in the project region.

Operating Basis Earthquake (OBE): The (OBE) may be expected to occur during the lifetime of the dam. No damage and/ or loss of service must happen. It has a probability of occurrence of about 50 % during the service life of 100 years. The return period is taken as 145 years. The (OBE) ground motion parameters are estimated based on a (PSHA). The mean values of the ground motion parameters of the (OBE) can be taken.

Construction Earthquake (CE): The (CE) is to be used for the design of temporary structures; such as coffer dams and considers the service life of the temporary structure. There are different methods to calculate this design earthquake. For the temporary diversion facilities, a probability of exceedance of 10% is assumed for the design life span of the diversion facilities. Alternatively, the return period of the

(CE) of the diversion facilities may be taken as that of the design flood of the river diversion. The MDE, DBE, OBE and CE ground motion parameters are usually determined by a probabilistic approach (mean values of ground motion parameters are recommended), while for the (MCE) ground motion deterministic earthquake scenarios are used (84 percentile values of ground motion parameters shall be used). However, for the MDE, DBE, OBE and CE also deterministic scenarios may be defined.

If reservoir-triggered seismicity (RTS) is possible; then the (DBE) and (OBE) ground motion parameters should cover those from the critical and most likely (RTS) scenarios as such events are like to occur within years after the start of the impounding of the reservoir.

The different design earthquakes are characterized by the following seismic Parameters:

1. Peak ground acceleration (PGA) of horizontal and vertical earthquake components.
2. Acceleration response spectra of horizontal and vertical earthquake components typically for 5% damping, i.e. uniform hazard spectra for (CE), (OBE), (DBE) and (MDE) obtained from the probabilistic seismic hazard analysis (mean values), and 84 percentile values of acceleration spectra for (MCE) obtained from the deterministic analysis using different attenuation models.
3. Spectrum-compatible acceleration time histories for the horizontal and vertical components of the (MCE) ground motion determined either from a random process or by scaling of recorded earthquake ground motions. The artificially generated acceleration time histories of the horizontal and vertical earthquake components shall be stochastically independent. To account for aftershocks, it is recommended to increase the duration of strong ground shaking.

In case of fault movements, similar estimates are required for the ground shaking. It appears that it is quite difficult for dam designers to get quantitative estimates of fault movements for the different types of design earthquakes. So, they are justified in such cases to use simplified load and analysis models that lead to a safe design, even if the load model does not comply fully with the real nature of the earthquake ground motion.

Considering the Appurtenant Structures of dams, these may be assigned to the safety level to which they belong with respect to the safety of the whole dam as follows:

Safety class 1: All elements related to the safe control of the reservoir, i.e. Bottom outlets and spillways are defined as safety-critical or safety relevant elements; they shall be designed for (SEE) and OBE (serviceability).

Safety class 2: All structures and components related to power production (penstock, power intake, powerhouse, tunnels, caverns, turbines, switchyard, transmission lines etc.), water supply, irrigation, navigation etc.; they shall be designed for (DBE) with high importance factor or according to earthquake building code as minimum requirement for the region under consideration.

Safety class 3: Other items which can easily be replaced/repaired when damaged and whose failure has acceptable consequences; they shall be designed for (DBE) or according to earthquake building code, [23], [24], [25] and [26].

6. Seismic Hazards and Their Impacts on Dam Safety

During the planning and design stage, safety of the dam may be ensured as far as seismicity is concerned by concentrating the study on certain seismic factors, which are the following:

Proximity of Site to fault Lines

The selection of the most appropriate type of dam may be affected by its distance from a known fault line. In a known fault zone, the possibility of existence of some branches from the main fault line should be given serious consideration. The Shih-Kang concrete gravity dam (Barrage) already discussed happened to be located on a branch of a main fault which had spectacular horizontal and vertical displacements during the 1999 Taiwan earthquake that resulted from the rupture of this fault causing failure of the dam. It is questionable; however, if any other type of dam could have endured this severe displacement, which had occurred in this case. Avoiding altogether such site for building a dam would have been the safest solution if the existence of such a fault was confirmed beforehand. Fault movement in the dam foundation or discontinuities in dam foundation near major faults can be activated, causing structural distortions [27].

In the case of Rubar Lorestan Dam at the Zagros Mountains in western Iran, the site was only 1.6km from the Saravand- Baznavid Fault. The main recent fault is thought to have moved about 50km in the last 3-5 million years implying a horizontal slip rate of 10–17mm/yr, and it was the source of frequent earthquakes of M=6 to M=7 earthquakes. The peak ground acceleration at the dam site was calculated using seven different attenuation formulae, and it varied from 0.52g to 0.61g. After long discussions as to whether a rockfill dam might be better able to resist fault movement than the rolled compacted (RCC) dam, but possibly it would be much more expensive than RCC dam as it would need spillway tunnels through the abutments whereas spilling water could have been routed over an RCC dam at little extra cost. The decision was to build the cheaper RCC dam even that the Rockfill dam is a little safer [28].

Liquefaction

Liquefaction is a phenomenon in which the strength and stiffness of a soil are reduced by earthquake shaking or other rapid loading. Liquefaction and related phenomena have been responsible for tremendous amounts of damages in historical earthquakes around the world.

Liquefaction occurs in saturated soils, that are soils in which the space between individual particles is completely filled with water. This water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together. Prior to an earthquake, the water pressure may be relatively low. However, earthquake shaking can cause the water pressure to increase to the point where the soil particles can readily move with respect to each other. The previous concepts apply to any earth retaining structures that is in contact with water and located in seismic regions, such as an earth fill dam. Liquefaction, therefore, is a serious potential problem for dams built on or with low density, saturated sands. In such cases, liquefaction potential may exist either in the embankment material itself, or in the foundation alluvial (in case the dam is resting on soil), and/or the abutment. The crest of the 40m high Lower San Fernando Dam settled 8.5 meters in the 1971 earthquake which had a magnitude of 6.6. The dam was built of hydraulic fill, which is particularly vulnerable to liquefaction, because of the low density of the fill. Fortunately, the water level was about 11m below the crest before the earthquake, but only 1.5m of badly cracked material remained after the event. 80,000 people living downstream of the dam had to be evacuated. The 8m high Sheffield dam failed completely in the magnitude 6.3 Santa Barbara event of 29 June, 1925. The dam and its foundation were silty sand, and some experts have blamed the failure on liquefaction of these materials. Krasnodar Dam in Russia near to the Black Sea is 11.5km long and built of hydraulic fill. It holds 2,914Mm^3 in a reservoir with an area of 413km^2. A seismic study was carried out by Swiss Experts who recommended improved drainage at a cost of 56 million USD. The cost of failure, if happened, would have cost an estimated 3 billion USD at the time (about 2000).

Tailings Dams have a particularly bad record with hundreds killed in various liquefaction failures in Chile. One example from the Dominican Republic is the 84m high Las Mejitas tailing dam, which holds 48 million tons of very acidic tailings at the Pueblo Viejo gold mine, only 35km from the Septentrional-Orient Fault Zone (SOFZ), where the estimated (MCE) at the site is 0.5g. It was reported that there was lack of adequate zoning in the dam and further studies were recommended for the dam in 2002.

Under seismic action, liquefaction of the tailings which would be denser than water, seems to be possible, [28], [29], [30] and [31].

Seiches

Some authors consider that reservoir oscillation due to ground shaking, and the following seiches are of lesser importance on dam safety than cases of ground shaking itself, which may cause vibrations in the dams and its appurtenant structures and equipment, or from fault movements in the dam foundation causing structural distortions, and also fault displacement in the reservoir bottom causing water waves or loss of freeboard; or even mass movements into the reservoir resulting from landslides causing impulse waves in the reservoir. Some other authors, on the hand, give examples of some serious cases of seiche which might have been caused by resonance of water in reservoirs that were disturbed by seismic activity acting on dams, [28].

A preliminary risk analysis was completed recently of two dams in Japan. One of them, a 131m high dam, settled 30mm in an M=7 earthquake in 1961. The probability of failure of the two dams as a direct result of an earthquake was shown to be negligible even though a seismo-genic fault passed only 250m from the right abutment of one of them. Moreover, the analysis indicated that the chance of seismic seiches and seismically induced landslides into the reservoir causing displacement wave and overtopping of the dam was also low. The calculated probability of dam failure from seismic seiches has been checked with previous dam failure statistics, and it looked that such probability was very low as compared to other modes of failures; as seen from Table 5.

Table 5: Estimated annual probabilities for the principal modes of failure.

Failure Mode	Annual Probability	Return Period (years)
Internal erosion	11.3×10^{-6}	88,496
Overtopping in typhoon	50×10^{-6}	20,000
Seismic seiches	4.3×10^{-6}	232,558
Landslide	9.3×10^{-6}	107,527
M & E spillway gate	13.2×10^{-6}	75,758
TOTAL	$\mathbf{88.1 \times 10^{-6}}$	**11,350**

Two cases of near failure as result of seiches are the Yuvaçik Dam, Turkey, in the 1999 earthquake and the Hebgen concrete-cored earth fill dam in Montana in the 1959 earthquakes. In the first case, the amplitude of the seiche was about 5m but the reservoir was not full at the time so there was no overtopping. If the reservoir had been full there could have been up to 1.68m overtopping at the abutments and 0.25m at the center of the dam. In the 35m high Hebgen dam, the reservoir was full at the time. This seiche was caused by fault movements crossing the reservoir rather than by ground shaking. A few minutes after the first shock, the caretaker had rushed to the dam and, in the moonlight observed the reservoir action from the high ground above the right abutment. The first waves had already overtopped the dam before he arrived. A few minutes later, another wave struck the dam with such momentum that water one-meter-deep ran uniformly over the crest for 10 minutes. Subsequently, the wave receded and seemed to travel to the other end of the reservoir. After 10 minutes it returned, and water flowed over the crest for another 10 minutes' period. This action was repeated and although the estimates of depths and durations are approximate, there can be no doubt that the water flowed over the dam at least four times causing some damage, but the dam did not fail. The magnitude of the earthquake was 7.5 to 7.8 with one of the main faults passing within 215m of the dam.

Landslide

Landslides can form real hazard to populations in mountainous areas, where relatively unstable masses of rock and earth may slide down in mass and hit communities living below causing real havoc by destroying houses and structures and burying people. Triggering of these landslides may be initiated by rainfall causing saturation of the soil mass and its sliding over the surface of a weak surface within. Such failure can also happen as a consequence of seismic ground shacking resulting from an earthquake, especially when such weak potential plain of failure exists [33] and [34]. If such landslide occurs into the reservoir of a dam, then there is a chance that overtopping, and failure of the dam will follow causing destruction and human fatalities downstream. Details and summary of landslides associated with dams and resulting fatalities are given by Professor David Petley from Durham University in a presentation given in the 2013 through an International Conference held in Padua in 2013 on Vajont (1963) incident entitled “Landslides and Large Dams”. The presentation examines losses associated with large dams in the last few years and shows in particular, that there have been over 500 deaths in landslide-related accidents during dam projects [35]. This presentation was followed after about one month by a blog written by the same author giving some more details [36]. In a paper entitled “Global Losses from Land Slides Associated with Dams and Reservoirs”, it is stated also that the risk of landslides can be exasperated by the seismic activity resulting from impounding large reservoirs known as Reservoir Induced Earthquakes (RTS). From the technical perspective, it may be said that seismic activity can profoundly alter the rates of activity of the landside, such that conditions that apply during a site investigation phase may no longer be current later on [37]. Research work, however, has also shown that seismically triggered landslides can affect dams. One particular study indicated that the behavior of an earth dam may depend on the probability of landslide slipping in its reservoir. Despite previous studies which have been done on the landslides without dynamic analysis, this study has investigated the effect of landslide impacting on earth dam during an earthquake. It showed that safety factor during an earthquake for earth dams decreased approximately by 40 percent, and the probability of landslide and its undesirable effects on the earth dams in the presence of reservoir water increased significantly [38]. Another study has concluded that moderate to large earthquakes can trigger landslides, and these landslides commonly cause a significant proportion of total earthquake damage. The ability to predict slope stability during earthquakes is especially important for seismic hazard analysis of dam projects [39]. Earthquake hazard is multi- sided for large storage dams. Recent earthquakes have demonstrated this in many such projects, which have been affected in many different ways, but those which are related to landslides alone can be listed among these, as in the following:

1. Rockfalls causing damage to gates, spillway piers cracking, retaining walls overturning, surface powerhouses cracking and puncturing, and various damages to electro-mechanical equipment, penstocks, switchyards, transmission lines, etc.
2. Mass movements of landslides and rockfalls into the reservoir causing impulse waves and overtopping of dams.
3. Mass movements blocking rivers and forming landslide dams and lakes whose failure may lead to overtopping of downstream run-of-river power plants or the inundation of powerhouses and the electro-mechanical equipment.
4. Mass movements blocking access roads to dam sites and appurtenant structures [40].

Landslide problem in relation to dam safety has been extensively studied on selected case histories treating it according to, landslide type, rock fall; including talus deposits, problems encountered due to interaction between dams and landslides, and mitigation measures to stabilize such as landslide.

USGS in its professional paper “Interaction of Dams and Landslides-Case Studies and Mitigation”, published in 2006 presented very good information on landslides [41]. Although this publication does not treat seismic triggered landslides with respect to dam safety, but it gives extensive general information on landslides in relation to dams as such.

Generally, it can be said from the foregoing that the probability of dams’ failure resulting from seismically triggered landslides is very remote. Failure probability of dams caused by landslides is shown to be 9.3×10^{-6} as already indicated from Table 5. If this probability is coupled with the probability of occurrence of an earthquake which is strong enough to cause such a landslide, then the resulting probability of failure of the dam becomes very remote indeed.

7. Reservoir Triggered Seismicity (RTS)

One important issue associated with the question of dams’ safety is the phenomenon of Reservoir Triggered Seismicity (RTS). This phenomenon is not fully-understood, but basically what happens is this; when a dam is built and the reservoir filled with water, the amount of pressure exerted on the earth in that area changes dramatically. When the water level of a reservoir is raised, the pressure on the underlying ground increases, and when the water level is lowered, the pressure decreases. This fluctuation can stress the delicate balance between tectonic plates in faults beneath the surface, possibly causing these plates to shift. Another factor is the water itself; when the water pressure increases more of it is forced into the ground, accordingly, filling cracks and crevices. This water pressure can expand those cracks and even create new tiny ones in the rock causing greater instability below ground. What is more is, as the water sinks deeper, it can act as sort of a lubricant for rock plates that are being held in place by friction alone. The lubrication can cause those plates to slip [42].

The sure fact is the dam cannot cause an earthquake all by itself. The risk factors, specifically unstable fault lines, have to be there already. With the right conditions in place, a dam can trigger the event earlier than would have happened naturally, and perhaps even increases its magnitude, which is why it is so dangerous to build a dam over a known fault.

A Study case of (RTS) may be cited here, as an illustration; this is the case of Danjiangkou reservoir. The study explains the various factors contributing to trigger the seismicity which had begun after impounding this reservoir in 1967 in Henan and Hubei provinces in central China. The Danjiangkou dam was built in 1958, and impoundment of the 174.5 × $10^{\mathbf{8}}$ $m^{\mathbf{3}}$ capacity reservoir began on 5 November 1967. The originally designed height of the dam was 97m, making the impounded reservoir one of the larger man-made lakes in Asia. As the source of Middle Route of South-to-North water diversion in China, the height of the dam was supposed to be increased to 111.6m starting in 2005 to be completed in 2010. Tectonically, there are four major Paleozoic deep regional crustal faults that crisscross in the reservoir area. They are overlain by about 4 to 5km thick sedimentary cover. Below the sedimentary cover, the crustal thickness varies from 33 to 34 km in the east, to about 37 to 41km in the mountainous area to the west. The deep crustal faults in the reservoir area are: Danjiang, Junyun, and Gonglu and Hanjiang faults. The study confirms that this tectonic setting is responsible for the (RTS) induced in the area. The increased load of the reservoir has caused compressive stresses on the bottom of the reservoir which can destabilize the underlying faults, and form approximately 150KPa tensile stresses in horizontal direction on the surface within the periphery of the reservoir, which might help to open small shallow fractures and promote the permeation of water into deeper rocks. This suggests that the induced seismicity in the reservoir area is mainly attributable to water and migration along the Danjiang and Junyun faults [43]. Other studies have also shown that the reservoir triggered an earthquake is linked to dams higher than about 100m, to large reservoirs (capacity greater than 500 x $10^6m^{\mathbf{3}}$), rate of reservoir filling, and to new dams of smaller size located in tectonically sensitive areas. This means that the causative fault is already near to failure conditions and so the added weight stresses and pore pressures propagation due to reservoir impounding, can trigger the seismic energy release.

1. The detection of the reservoir induced seismicity may be performed in two phases: The first phase includes study of historical seismicity and surveys of the reservoir and surrounding geological structures, aiming at identification of possible active faults.
2. Second phase is carried out, starting at least one or two years prior to impounding, with the installation of a permanent network of seismometers and precise levelling beacons and use of instrumentation to detect active fault movements, in addition to carrying out reservoir slope stability studies.

In Table 6 some examples of dam sites are presented, where induced earthquakes with magnitude higher than 5 on the Richter scale have occurred.

The reservoir triggered an earthquake differs from the natural earthquake as those are likely to occur, if ever, within the first 5 years after the filling of the reservoir or when the reservoir reaches the maximum level. These earthquakes have a shallow focus, and their epicenters are close to the dam sites or reservoirs [44].

Table 6: Examples of dams with induced seismicity [44].

Dam	Country	Type	Height (m)	Reservoir volume (x 10^6 m^3)	Year of impounding	Induced seismicity		Prior seismicity
						M	year	
Marathon	Greece	Gravity	63	41	1930	5	1938	moderate
Hoover	U.S.A.	Arch gravity	221	36703	1936	5	1939	---
Kariba	Zimbabwe/ Zambia	Arch	128	160368	1959	5.8	1963	low
Haifengkiang	China	Buttress	105	10500	1959	6.1	1962	aseismic
Koyna	India	Gravity	103	2708	1964	6,5	1967	low
Kremasta	Greece	Embankment	165	4750	1965	6.3	1966	moderate
Roi Constantine	Greece	Embankment	96	1000	1969	6.3		moderate
Oroville	U.S.A.	Embankment	236	4298	1967	5.7	1975	moderate
Nurek	Tajikistan	Embankment	330	11000	1972	5	1977	moderate
Tarbella	Pakistan	Embankment	143	14300	1974	5,8	1996	low
Aswan	Egypt	Embankment	111	163000	1974	5.3	1981	aseismic
Polyphyton	Greece	Embankment	112	2244	1974	6.7	1995	aseismic
Morns	Greece	Embankment	126	640	1961	----	----	aseismic

8. Seismic Instrumentation for Dams

Large dams are particularly sensitive to earthquakes. They are usually built in valleys, which exist because active erosion is taking place. This implies there has been recent uplift produced under compressional tectonic force reverses or thrust faults, so reverse or thrust faults dip under the up- thrown block. Many dams, therefore, have an active fault dipping under them. Reservoirs created by dams can also trigger an earthquake if certain conditions are met, i.e.; if the reservoir is deep enough and the water load is appreciable, the ground underneath is saturated and hence the high groundwater pore pressure decreases the effective strength of the rock under the reservoir, the area is already under considerable tectonic stress, and faulting exists, and in many cases, this activity is increased by raising and lowering of the water level. Such reservoir triggered seismicity will have its repercussions on the dam and its appurtenant structures in the same way as natural earthquakes [45]. It is of vital importance that, seismic events whichever their source, must be studied and their impacts on dams analyzed. Any meaningful dynamic analysis of dams which reliably predicts the behavior of dams during very strong ground shaking requires sufficient data on the spatial variation of ground motion acting on the dam's body, and such data may be collected by seismic instrumentation. In such a capacity effective seismic instrumentation and monitoring of dams can help to monitor the safety of dam and its proper maintenance and up-keep. It can also considerably contribute to the overall activities for seismic risk reduction and facilitates response studies that lead to improved understanding of the dynamic behavior and potential for damage to structures under seismic loading. It is necessary; therefore, that responsible dam personnel and higher staff have proper understanding of the work of instrumentation, their maintenance and relevant records/data and their analysis. The specific purposes of dams seismic monitoring may be summarized by the following:

1. Determination of the location of earthquake epicenters relative to dam location and their depth.
2. Definition of main earthquake parameters: magnitude, frequency characteristics and some indications of focal mechanisms.
3. Prediction of the mode of occurrence of future earthquakes.
4. Provision of data on the dynamic behavior of the dam body for the purpose of objective evaluation of its functioning immediately after the occurred earthquake.
5. Verification of design parameters by the actual behavior of the dam body under an earthquake.

To achieve the aforementioned goals, it is necessary to investigate and monitor the dam site by means of various seismic instruments, so it is desirable to install seismic instruments on the dam itself and at near locations for this purpose. The various types of instruments required are:

1. Strong-motion accelerographs for recording potentially destructive ground shaking and resulting dam vibrations.
2. Sensitive seismographs for determining the local seismicity. A minimum of two strong-motion accelerographs should be installed on the dam and a minimum of two should be installed in the immediate vicinity of the dam. Each accelerograph should record the three components of motion, should have a natural frequency of approx. 20Hz, a recording speed of approximately 1cm/s. The sensitive seismographs are intended to record the local seismicity in the vicinity of the dam site before construction, and to detect any changes in seismicity during reservoir filling. A vertical component seismometer (1Hz-5Hz) with visual recorder and approximately 10,000 magnifications at 1Hz is recommended.
3. Seismic data analysis system.
4. Magnetic tape play-back system.
5. Digital time marking system.
6. Digital event recorder.

The instruments shall be suitable to record the earthquake detail up to magnitude 8.0 on Richter`s scale. The microearthquake may contain frequencies up to hundreds of Hertz (Hz), while an earthquake recorded in a seismic net may contain frequencies between 1 and 20Hz. So, it is very difficult to achieve such a dynamic range in a single recording system with an equal response to all the frequencies, [46]. The sketch in Figure 3 indicates recommended dam seismic instrumentations and their arrangement; at the crest, abutments, foundations of the dam and free field in the vicinity. In this way, full information on the ground shaking at the site in addition to a full dynamic response of the dam may be obtained and then be correlated with other available data from seismic national or international networks [45]. Safer dams with respect to earthquakes can be designed and built now with more confidence. Technologies for designing and constructing dams and appurtenant structures that can safely resist the effects of strong ground shaking are available. This includes in addition to developed seismic criteria and analytic methods of design, an array of modern seismic instrumentation as a good asset to analyze the dynamic response of dams during earthquakes, to learn more on their behavior in such events and draw more lessons for future works.

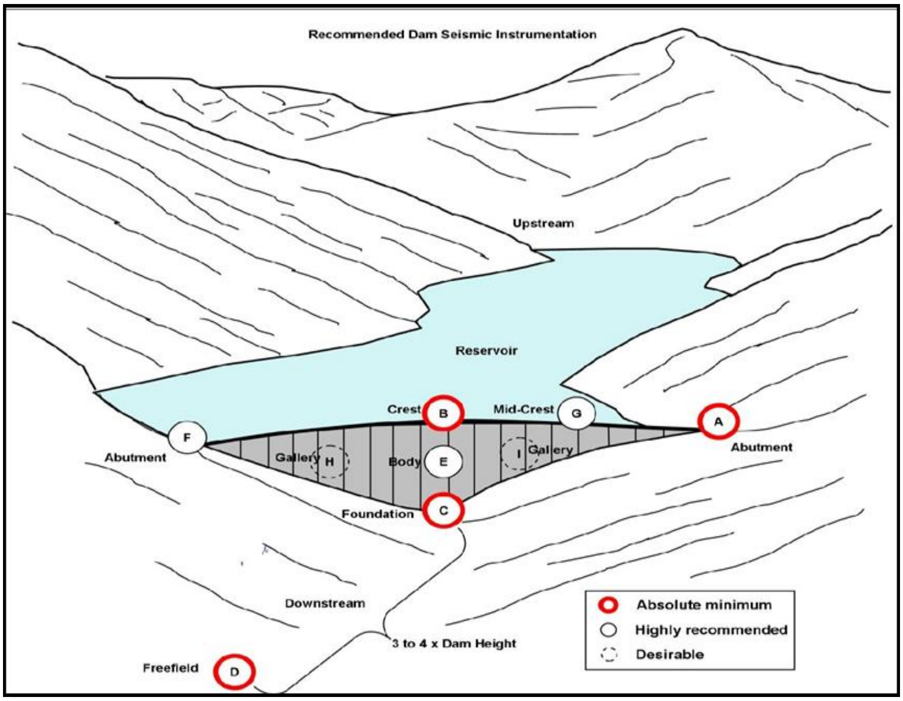


Figure 3: Recommended dam seismic instrumentation [45].

9. Inspection Procedures after Seismic Events

In order to assess the safety of dams affected by earthquakes, thorough investigation is usually required by the dam safety authorities immediately after the event. According to the American Army Corps of Engineers (USACE), this assessment is to be done in certain recommended procedure, which is done by carrying out initial inspection and evaluation survey, followed by Post-Earthquake Inspection (Engineering).

Inspections and Evaluation Surveys

The design of dams for earthquake loading is hampered by the infrequent opportunity to compare actual performance with the design assumptions. Significant damage is unlikely to occur if peak ground acceleration (PGA) is below 0.1g; nevertheless, structural integrity cannot be assumed if earthquake loadings occur below the design criterion. For example, earthquakes have occurred in places where significant seismic activity was not expected, and where an earthquake induced loads may therefore have not been adequately considered in the design of older structures.

Many types of structural damages can be induced by ground motion from earthquakes. Any post-earthquake change in appearance or functional capability of the dam should be evaluated and reported. The inspection engineers should look carefully into any such damage, whatever small it may be, and report any of the following:

1. Symptoms of induced stresses evident by cracks in concrete dams or earth embankments.
2. Misalignment of hydraulic control structures or gates, and possible loss of function.
3. Cracks in service buildings or pavements.
4. Rock falls from the adjacent slopes on the dam.
5. Loss of freeboard induced by dynamic loading on earth dams or levees, which may result from settlement.
6. Dynamic loading on earth dams or levees may also cause localized liquefaction within the embankment sections or earth foundations leading to deformation of the embankment.
7. Hydrologic changes also can indicate the potential for subsurface deformation, such as abrupt changes in static water levels in wells and increased turbidity.
8. The appearance of surface seeps can indicate the formation of new seepage paths within the foundation or embankment section.
9. Ground motion-induced landslides may occur in susceptible areas of reservoir rims, causing embankment overtopping by waves and serious damage.

Inspection and Evaluation Programs

The following characteristics can be useful for prioritizing the sequence of structural inspections and damage evaluation:

1. Significant Damage to Structures in the Vicinity: If the project is located within the vicinity of an earthquake that results in significant structural damage, an engineering evaluation and inspection should be carried out by a component team from the local authority, while the project operations\` personnel will conduct and document an initial damage survey inspection of the project without delay. The project's emergency operations plan should be invoked. Actions during emergency operations should include consideration of the effects of potential aftershocks.
2. No Significant Damage to Structures in the Vicinity: If an earthquake is felt at the project during normal working hours but causes no or insignificant damage to structures in the vicinity, project operation personnel should make an immediate inspection. If the event occurs after normal working hours, the project operations manager will determine the need for the immediacy of the inspection but in no case will it be delayed beyond the beginning of the following day. This inspection should determine and document, (i) whether there is evidence of earthquake damage or disturbance, and (ii) whether seismic instrumentation, if present, has detected ground motion, then a Safety Emergency Alert should be issued and follow reporting procedures.

Combinations of Magnitude and Distance

1. The following criteria presented by USACE and known by "Earthquake Inspection Criteria Simplified Model" are listed below and illustrated by Figure 4 allow operations managers to assess the immediate need for site inspections and evaluations based on earthquake magnitude and proximity to an earthquake epicenter. If the magnitude and distance fall within the specified ranges, project operation personnel will conduct an inspection and check all the seismic instruments and alarms, if any are present at the project. If no seismic instruments are located at the project, then earthquake magnitude and epicenter data can be obtained from regional or international seismological centers such as; The Global Seismographic Network (GSN), The International Seismological Centre (ISC) or The European Mediterranean Seismological Centre (EMSC).
2. If any of the seismic alarms indicate a (PGA) of 0.05g or greater, or if damage is identified, then the inspection results performed by the project staff will be reported immediately to the designated personnel within the next higher engineering authority who will carry out themselves a more comprehensive inspection. The USACE criteria, aforementioned, for the need or otherwise to carry out the inspection is stipulated in the following ranges and illustrated in Figure 4:

- Magnitude 4.5 through 4.9 within 16 km (10 mi).
- Magnitude 5.0 through 5.9 within 80 km (50 mi).
- Magnitude 6.0 through 6.9 within 120 km (75 mi).

- Magnitude 7.0 through 7.9 within 200 km (125 mi).
- Magnitude 8.0 or greater within 320 km (200 mi).

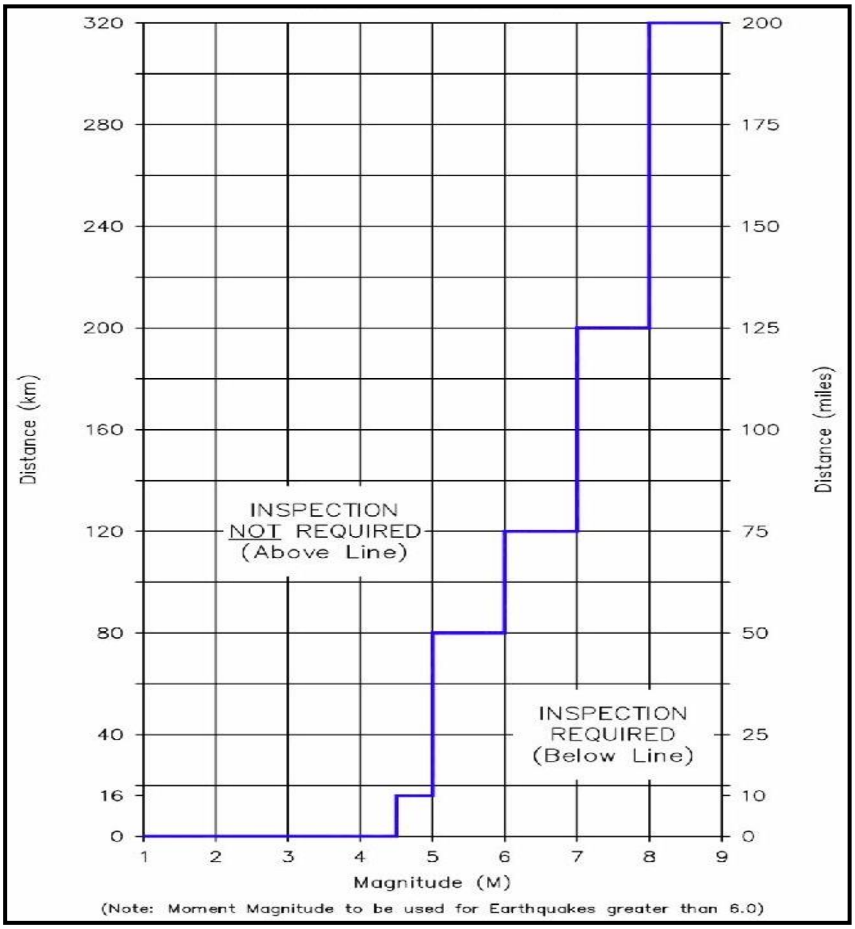


Figure 4: Earthquake Inspection Criteria – Simplified Model [47].

Analysis of Results and Final Report

It is of utmost importance that all compiled inspection reports be studied carefully and analyzed in the light of, but not limited to:

1. The physical characteristics of the earthquake event, including the magnitude of the earthquake, the distance between the epicenter and the project, the peak horizontal ground acceleration (either calculated or measured at the site).
2. The condition of the structure or embankment as documented during inspections.
3. The structure or embankment performance history. Whenever feasible, on-site instrumentation data is incorporated into the post-earthquake safety evaluation programs to fulfill inspection.

This Analysis may indicate the need to install in the future additional special types of instrumentation to be incorporated in selected structures to improve measurement of forces, pressures, loads, stresses, strains, displacements, deflections, or other conditions relating to damaging and structural safety and stability in case of an earthquake.

The final report shall not fail to include also the results of inspections of features not visible on the surface, such as conduits, tunnels, and galleries, which should have been performed with particular attention as damage or differential movement of these features could lead to delayed adverse effects. Moreover, inspections of underwater project features should be performed to include positional surveys of structures and/or channels as deemed necessary.

Summaries of the various instrumentation data and those of accelerometers or other types of strong motion instruments, if they have been installed, together with other observations' data for each inspection should be also appended to the final report for permanent record and reference purposes.

This final report must be viewed as of great importance and value as it will form the basis for major remedial work when required. The report should contain recommendations for remedial work when appropriate. For structures incurring no damage, a simple statement to this effect will be given, unless seismic instrumentation at the project is activated [47].

Dynamic Analysis for Dams After Earthquakes

For important dams, where great risks exist for human lives and material losses, dynamic response analysis is worth doing. The United States Bureau of Land Reclamation (USBR), in its Safety Evaluation of Existing Dams (SEED) Manual, presents guidelines and procedures to assess the earthquake stability of dams. In areas of low and infrequent seismic loading, the initial analyses are often conducted using simplifications and conservatively selected seismic shortcuts data and assumed properties. If this conservative analysis shows the dam to be safe, no further work will be required. In areas of more severe and/or frequent seismic loading, and in the case where an initial simple analysis does not demonstrate the dam to be safe, more sophisticated analyses are required as explained below.

A. Dynamic Stability (Deformation) Analyses:

Generally, these analyses incorporating the time dependence of the ground acceleration and the dynamic response will be conducted (i.e., more sophisticated than Pseudo static) for the various types of dams as follows:

1. Embankment Dams: The initial step will be a simplified analysis utilizing the NRC (Nuclear Regulatory Commission) response spectrum. Results to be obtained include the following:

i. The permanent displacements along assumed failure surfaces which extend through the top one-fourth, the top one-half, and the full height of the embankment and result from the critical MCE's (maximum credible earthquakes) and/or approximate probabilistic earthquakes.

ii. The epicentral distances of magnitude M = 6.5, 7.5, and 8.25 events which would cause a 3-foot permanent deformation along a failure surface extending through a critical section of the embankment.

More sophisticated second phase analyses would then be conducted as needed. The more sophisticated analyses are usually staged (i.e., be progressively more exact) until either the dam is demonstrated safe or determined to be unsafe which is usually more difficult.

2. Concrete Gravity Dams: The initial step will be a simplified response spectrum analysis utilizing the Nuclear Regulatory Commission (NRC) response spectrum. Local site effects are not considered in the determination of the spectral amplitudes. Results which are to be obtained include the following:

i. Peak stresses in critical elements, factors of safety for overturning and sliding, or analysis by energy methods if the factor of safety is below 1.0 resulting from the critical MCE's and approximate probabilistic earthquakes.

ii. The epicentral distances of magnitude M = 6.5, 7.5, and 8.25 events which cause critical elements to become overstressed, or yield factors of safety of 1.0 for overturning and sliding.

3. Arch and other Concrete Dams: The initial step will be a simplified response spectrum analysis. Details of the analyses vary from case to case. Results to be obtained include, as applicable, the ones listed for concrete dams above.
4. Appurtenant Structures: The level of analysis varies from a simple qualitative assessment to more detailed response spectrum analyses depending on the importance of the appurtenance to the overall safety of the dam.

B. Liquefaction Analyses: Liquefaction analyses will be conducted for all foundations and embankments where an initial assessment indicates the presence of potentially liquefiable materials. The initial analyses would be by simplified methods. Further, Phase II analyses would be performed to the extent required.

C. Fault offsets through the Dam and/or Abutments: The effects of fault offsets would be assessed on a case-by-case basis.

D. Seiche: The effects of seiche arising from ground accelerations (i.e., not faulting or land sliding) in the reservoir considering oscillations perpendicular to and parallel to the dam would be investigated. If the earthquakes under consideration have significant energy content at these periods, then a simplified modal superposition analysis will be conducted, and the resultant wave amplitudes estimated. Further second phase analyses would be accomplished as needed including the effects of overtopping of the dam.

E. Landslides and Fault Displacement Waves: The effects of landslide and faulting in the reservoir bottom have to be investigated.

10. Rehabilitation of Embankment Dams after Seismic Events

Although many embankment dams have failed during earthquakes, many more have survived these events but sustained damages of varying magnitudes and remained operational after rehabilitation. Studying these cases indicates that recent dams which were designed and constructed with modern techniques have behaved much better than the older dams. Lessons learned from dam failures, and incidents are beneficial in avoiding the entire possible shortcomings, while rehabilitation works carried out indicate possible solutions in similar conditions. The rehabilitation works of embankment dams after earthquakes are discussed here describing in general the type of damage(s) and suggesting guidelines for treatment. Each dam, however, has to be looked at as a unique case and the exact treatment implemented accordingly.

1. Seismic Evaluation

Proper inspection of dams for any evidence of displacement, bulging, depressions or undue settlement, tilting, slope instability, cracks, seepage, leakage, erosion and improper functioning of drains and relief wells, is required immediately after a seismic event. Any of these conditions, if corrective measures are not taken, can ultimately lead to failure of the dam. Surface cracks on the crest or near the embankment or embankment abutment contacts can be an indication of settlement of the embankment, and if severe enough, a path for leakage can develop along the contacts. Therefore, these crack locations must be thoroughly examined, and the following procedures should be adopted for restoration works. In seismic evaluation of a dam, it has to be ensured that sufficient defensive measures are incorporated by ensuring good quality control, adequate compaction of materials, foundation and abutment integrity, ample freeboard, provision of gentle u/s and d/s slope.

2. Repair and Restoration Measures for embankment Dams

Damage to embankment dams varies between, settlements, cracking, increased seepage and liquefaction of parts of dam or its foundation. Settlement may be compensated by additional fill or constructing parapet. Other damages may be treated in prescribed ways. In the following some information is given on such treatments:

a. General treatment of cracks

Wherever some sort of cracking on dam’s body or separation at its abutments are visible, they should be investigated in detail, and appropriate remedial works should be carried out. If the cracks extend some distance along the crest, then test pits are excavated at suitable intervals to check their depths The trench shall encompass the whole crack and extends to its full depth and then filled and compacted with the appropriate material. Further treatment is carried out according to the seriousness of the crack. An example may be cited from Takami dam in Japan following the Tokachi-Oki earthquake (M= 8.3) on September 26, 2003. A special inspection after the earthquake found longitudinal cracks parallel to the dam axis at the crest of the dam, refer Figure 5. The cracks were up to 160m long in the dam axis direction. The maximum width of the cracks at the surface was 50mm. An exploratory pit at the locations of the widest crack was then excavated to measure the depth of the crack. The results confirmed that the cracks remained within the crest protective layer with a thickness of about 90cm without reaching the core zone. The excavated pit was filled in immediately after the survey, and to prevent seepage by rainfall from expanding the cracks to the core, the entire cracked area was covered with impervious sheets. One follow-up survey of the crack depth at the crest was carried out later on as a check. The results confirmed that as in the case of the previous survey, none of the cracks extended beyond the protective layer. The locations of the cracks inside the protective layer were repaired by backfilling them with identical material, refer Figure 6. During the backfilling, density measurements inside the protective layer were done to control compaction so that the density would be the same as that before damage [49]. In case the cracks penetrate into the core, it is suggested to excavate a trench deep enough to expose the top of the impervious core, the trench to be further dug for 0.5m depths in the impervious material of the core. The side slopes of the trench are kept at 0.5:1 and bottom width be kept as 1.0m. The trench in impervious core is to be filled with water and allows water to seep through. If, at all, any sign of wetness or oozing is found. The trench is back filled with mixture of impervious soil and 2% to 3% of low-grade bentonite to be laid and compacted with hand- held pneumatic tempers at 2% of optimum moisture content and up to a depth of 1.5m. The upper 1.5m layer over core material is backfilled with semi-pervious type of soil with compaction by pneumatic temper. The crest is reinstated to its original shape, and riprap is then re-laid as per designed section. After refilling the trench, 3m c/c holes be drilled and grouted with clay bentonite slurry at a low pressure if warranted, Figure 7 [50].

b. Treatment for depression and protrusions on d/s face.

If the depressions and/ or protrusions are up to 200mm or less, the downstream slopes need not to be disturbed. In case the depressions are of larger magnitude, all material(s) in loose pockets should be removed. The face is to cut to normal to slope or at about 20°; as convenient. Suitable material(s) free from organic material shall be selected. In choosing material(s) for sand and gravel fill, the material(s) shall be well grade. The cut surface shall be moistened, and fill material shall be placed in about 30 cm thick horizontal layers. Each layer is to be compacted by some pneumatic tamper till its density is about 90 percent of dry density of the material. The thickness after compaction become about 20cm. Water content should be uniform throughout the layer to be compacted, and it should be as close as possible to that content which will result in the maximum densification of the material to be compacted. In general, this water content will be slightly less than the optimum water content as determined by Proctor compaction test.

c. Treatment for sloughing

Treatment for sloughing of downstream face is carried out by removing all undesirable vegetation. A previous blanket or filter material is first laid as buttress on the face and extended up to the toe filter. Longitudinal cracks observed on the downstream slope of the dam are to be excavated in the form of a deep trench having side slope of 0.5:1 and bottom width of 1.0 m keeping the crack in the center. The trenches are to be backfilled with selected soils with proper rolling and watering. The existing disturbed pitching on d/s slope is to be re-laid to the designed section for rain protection.

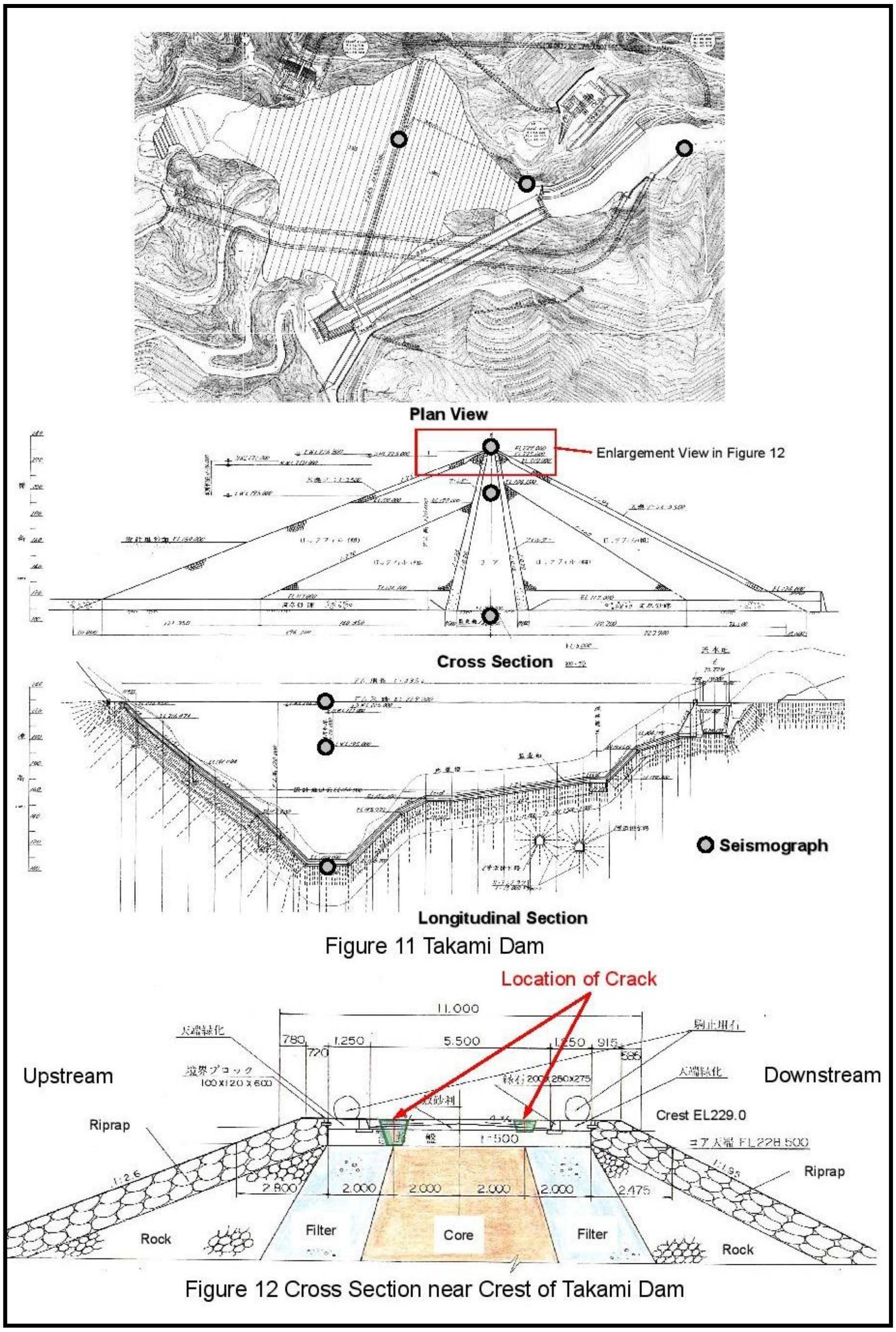
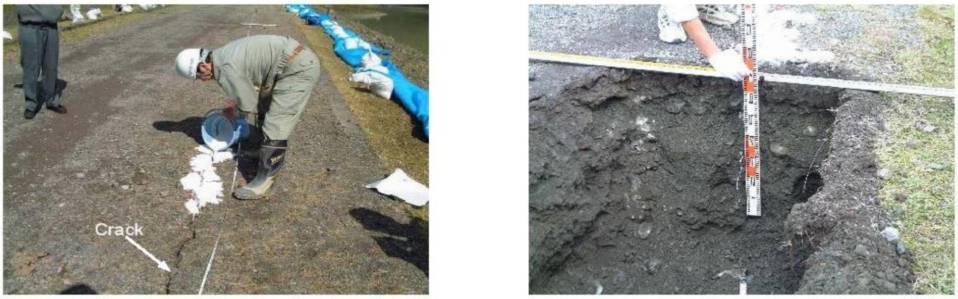
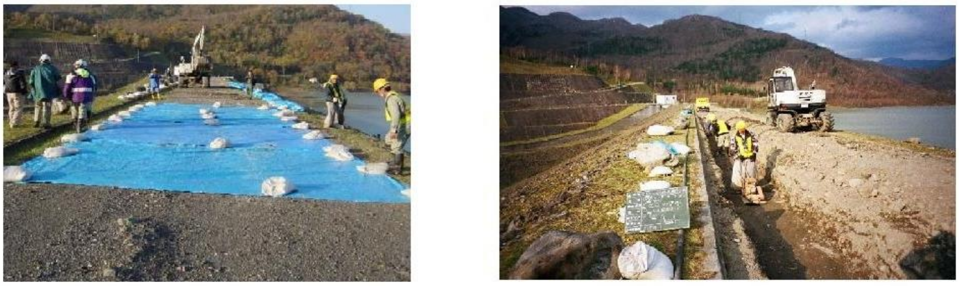


Figure 5: Takami dam and location of cracks at the Crest [49].



Crack at Crest of Dam

Survey of Crack Depth by Explotatory Pit



Protection of Cracks at Crest

Repair Work of Crack Location

Figure 6: Preparation for Filling the Crack on the Crest of Takami dam [49].

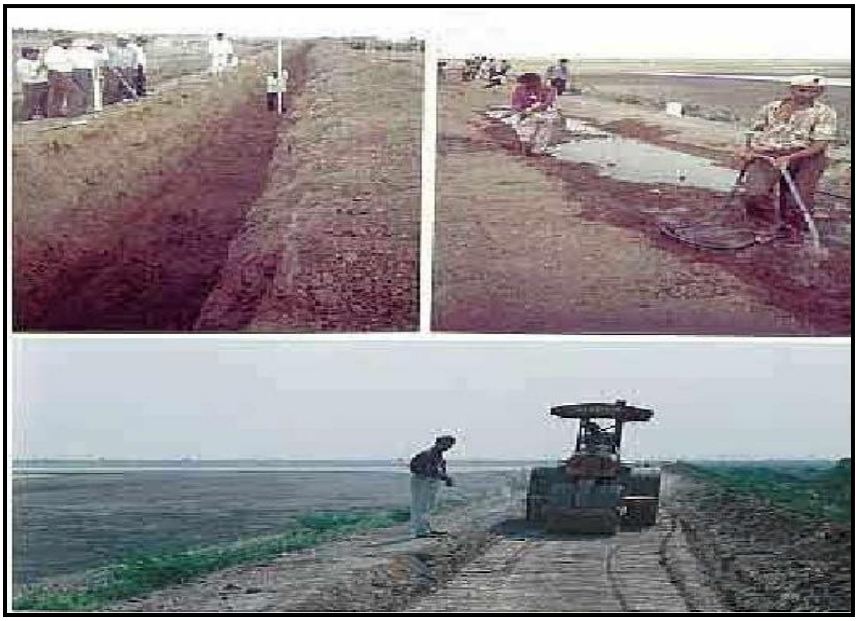


Figure 7: Treatment of longitudinal deep crack at Dam Crest [50].

d. Treatment for longitudinal crack and slip in u/s slope

Where the u/s slope has sloughed down and bulged due to longitudinal slips and settlement in the earthwork and pitching; the following treatment is to be followed: To remove first all the pitching work and the damaged earth on the upstream; then to excavate and remove the earthwork in casing sections in benches of 2 to 3m in depth and about 4 to 6m in width to facilitate rolling and watering in stages. These benches are to be excavated step by step, thus clearing the casing zone of all cracks. Next step shall be to complete the earthwork of the casing zone by taking selected materials from the borrow pit area, backfilling the excavated portion, after proper rolling and watering. Then after bringing the u/s slope to the original levels, pitching is to be duly completed.

e. Treatment of sliding in the upstream protective layer

The case at the Makubetsu Dam, an earth fill dam in Japan, with a height of 26.9m, completed in 2004, may serve as an example. In this case, the maximum accelerations of 173.1gal (dam axis direction) and 251.6gal (stream direction) were recorded in the dam foundation and at the crest, respectively. The epicentral distance of the dam was 140.6km. The sliding of the protective rock layer on the earth blanket placed on the left and right bank slopes upstream of the dam body occurred at two locations; on the left bank, length of the first was 20m, while length of the second was 23m, these are indicated by numbers 1 and 2 in Figure 8. In addition to two more locations on the right bank, the first was 80m long and the second was 30m long, and indicated by the numbers 3 and 4, respectively, in the same Figure 8. The topmost end of the sliding was directly above the water level during the earthquake. The location of the maximum sliding is near the spillway on the right bank side, and the maximum settlement of approximately 90cm was confirmed by the shape of the slope, and a level difference of about 40cm was formed at the top edges of the deformation, refer Figure 8. Close up of photos of the sliding are shown in Figure 9. After the earthquake, the dam management performed an excavation survey confirming that the sliding was limited to the protective rock layer (approximately 2m) and did not extend to the interior of the earth`s blanket. And the part of the protective rock layer, that became thinner because of the sliding, was thickened by placing more rock materials to temporarily repair the sliding. The pore water pressure inside the dam body rose slightly at almost all parts of the upstream side zone, and it had almost peaked at the time of the survey. The values obtained later by a regular inspection were stabilized, and almost no abrupt change was found. The seepage was increased slightly due to the earthquake, but it stabilized later. The dam body displacement in the upstream- downstream direction near the center of the crest is approximately 2cm on the upstream side, and the settlement is about 3cm.

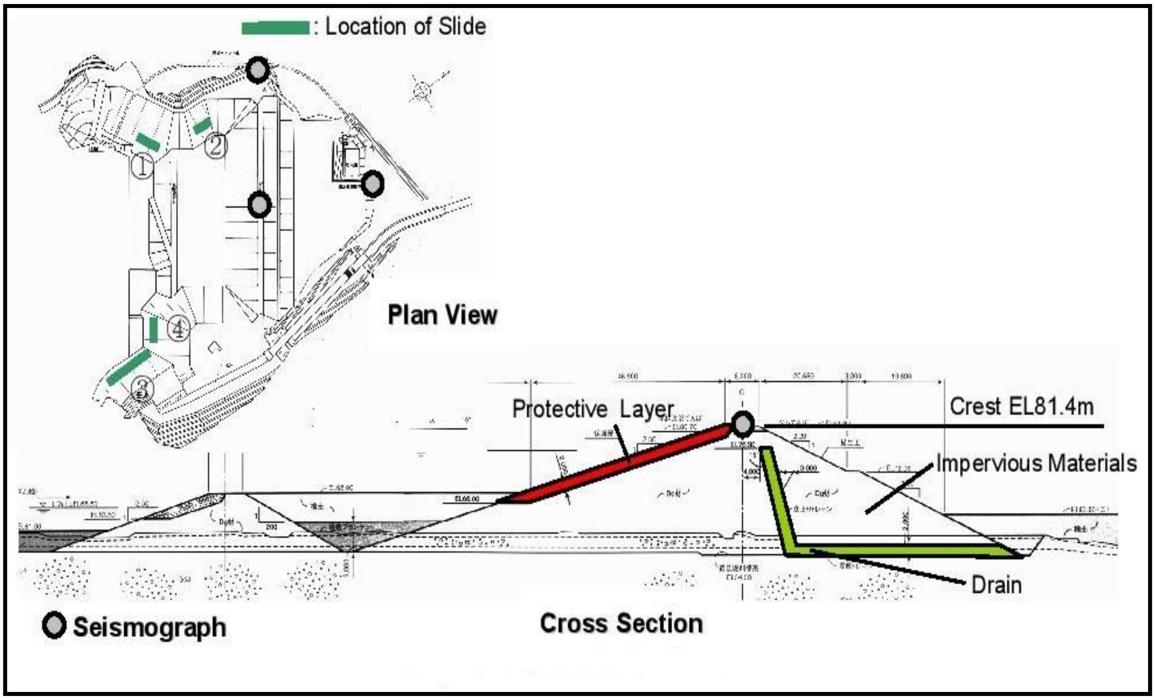
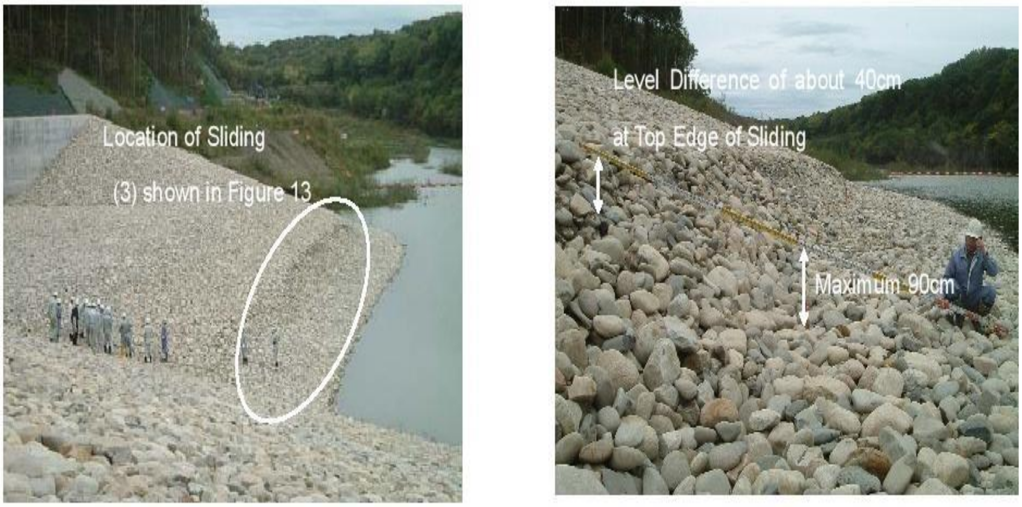


Figure 8: Makubetsu Dam layout and cross section showing damaged areas [49].



Sliding of Protection Rock Layer of Makubestu Dam

Sliding of Protection Rock Layer of Makubestu Dam (Enlargement)

Figure 9: (Upper) Layer of Makubetsu Dam and location of slides. (Lower) views of slides [49].

f. Treatment for seepage

Where heavy seepage is noticed in dams after earthquakes, the complete wet d/s area is to be cleared of any vegetative growth, particularly the toe drain area. The seepage water shall have to be channelized, and suitable basins created to measure the seepage water by providing adequate V-notches at suitable locations. The seepage is to be properly monitored, particularly, with reference to the reservoir levels.

In order to assess the final quantity of seepage and also the zones and the area where the seepage is occurring, whether through the body of the dam or below the foundation levels; the following measures are suggested: Piezometers may be installed on the d/s and also in the body of the dam. Monitoring of these would indicate the nature of seepage, whether through the foundation or the body of the dam. In case the seepage is mainly in the foundation area, it will be necessary to provide adequate treatment either by restoring to cut off measures to be suitably decided after the seepage details are available. Remedial measures for seepage such as upstream clay blanketing, downstream toe loading, etc. may have to be carried out in case the seepage is found to be excessive and concentrated at a particular location at the downstream toe. The seepage collected in the seepage drain at the toe of the dam should be measured over a V-notch placed at the exit of the dam. In no case, the seepage should be more than the acceptable limits prescribed by the design calculations.

g. Strengthening of slopes.

If a section needs strengthening or flattening of slopes, this should be done by placing coarse pervious material. On the u/s face, rock-fill material may be used, if possible. On the d/s face, if only limited placement is needed, once again it should be done by coarse pervious material. However, if substantial placement is required, the quantity of pervious material required can be reduced by placing a chimney drain connected to a horizontal drainage filter blanket and rock toe at the end. Beyond the chimney drain, any clean fill material can be used to give the required weight. Downstream slope should also be pitched to protect it from rain cuts.

h. Treatment for liquefaction

Where the dam is damaged severely due to liquefaction, the top layer or more as the case may need, are to be removed from the dam foundation area. The excavated layer should be replaced by well-compacted semi- pervious material upstream of the cutoff trench, and well graded compacted pervious material downstream of the trench. The cutoff trench should be carried down to the rock or hard sub-surface to provide a positive cutoff. Alternatively, it may be necessary to densify the foundation material by hammer driven piles on a 3m hexagonal grid through the full depth of the foundation. This could be done through 15cm minimum diameter pipes with a flat bottom, which could be extracted, and the hole filled back with pea gravel. It will be found convenient to use a flat sliding steel shoe at the pipe bottom. This treatment will have to be done first in a test area and post-treatment N-value compared with those before the treatment. Spacing of the pipe or its diameter could be suitably adjusted to achieving the desired N-value. The SPT values should be aimed at 20 or more, which will be indicative of no further liquefaction. According to Indian Standards IS: 1893-2002, the recommended N-values are given in Table 7.

Table 7: N-values with depth.

Depth below ground level	≤ 5 m	10 m
Minimum acceptable N value, Zone IV and V	15	25
Minimum acceptable N value, Zone III	10	20

Note: For intermediate depths, linear interpolation may be used

Where there are successive slides over the full height of the dam slope, it would be necessary to remove all the disturbed soil material from upper most slip face to the required depth. The removed portion can then be rebuilt with a homogeneous low permeability soil and properly compacted to bring the section to its original dimensions.

11. Upgrading of Concrete Dams for higher resistance to Seismic Events

Concrete dams, generally speaking, have fared much better than embankment dams as far as their resistance to earthquakes. But new studies have shown that they could fail under the effect of ground shaking and acceleration greater than what they have experienced. This has necessitated upgrading their designs and performing additional construction works to shore them up against greater expected events. Case studies on eight dams that were strengthened to improve their resistance to seismic shaking are presented in paper authored by Kenneth Hansen, and Larry Nuss [51] and [52]. All these dams are located in the earthquake prone western United States and Canada. The studied cases are three concrete arch dams, three multiple-arch dams, and two slab-and-buttress dams. The basic information on the dams is contained in Table 8.

These seismic upgrades were based on the knowledge and analytical tools available at the time. Modifications to these dams in order to improve their seismic performance were made mainly on the basis of 3-dimensional (3D) linear finite element analysis (FEA), which produced tensile stresses determined to be in excess of the predicted tensile strength capacity of the in-place concrete when the structure is subjected to the Maximum Credible Earthquake (MCE). Only three cases are described briefly here for illustration while the full scope can be found in the aforementioned references.

Table 8: Study cases of dams modifications to strengthen them against seismic events impacts.

Dam Location	Date Built Modified	Height feet (m)	Crest Length feet (m)	Dam Type	Modification	Design EQ	Design PHGA (g)
Big Bear Valley Big Bear Lake, CA	1912 1989	92 (28)	360 (110)	Multiple Arch Mass concrete	Mass concrete infill between buttresses	M 8.3 at 16 km	0.71
Gibraltar Santa Barbara, CA	1920 1990	169 (52) 194 (59)	600 (183)	Arch RCC buttress	RCC buttres downstream	M 7.5 at 2 km	0.68
Stewart Mountain Phoenix, AZ	1930 1991	212 (65)	583 (178)	Arch	Full height post-tension cables	M 7.5 at 15	0.34
Littlerock Palmdale, CA	1924 1994	175 (53)	720 (219)	Multiple Arch RCC	RCC infill between buttresses	M 8.0 at 2	0.70
Weber Placerville, CA	1924 2002	89 (27)	354 (108)	Triple Arch RCC	RCC stabilized sides of central buttresses	M 6,5 at 18 km	0.3
Stony Gorge Willows, CA	1928 2009	139 (42)	868 (265)	Slab & Buttres	Concrete walls and struts between buttresses	M 6.5	0.71
Seymour Falls Vancouver, BC	1961 2007	98 (30)	771 (235)	Slab & buttres	Inclined post-tensioned anchors for buttresses. Two concrete walls in bays on either side of spillway.	M 6.5	0.50
Big Tujunga Los Angeles, CA				Arch	Concrete buttres down stream	M 7.5 at 7 km	1.1

1. Bear Valley Dam

The Bear Valley Dam is located in the San Bernardino Mountains about 80 miles east of Los Angeles. It was the first concrete multiple arch dam to be upgraded to improve its seismic performance. The dam was completed in 1912 as a 92 foot (28m) high, 360 foot (110m) long multiple arch dam. In the 1980s, there were concerns over the structural adequacy of the dam with respect to severe earthquake shaking as well as overtopping. Two MCEs were considered for the dam rehabilitation. The most critical shaking was determined to be a M=8.3 earthquake caused by a rupture on the San Andreas Fault; 10 miles away with a PHGA of 0.45g and a bracketed duration of 35 seconds (duration between the first and last peaks of 0.5g or greater). The structural strengthening was accomplished by infilling the arch bays with conventional mass concrete in order to convert the multiple arch to basically a gravity dam. The downstream slope was formed at 0.25V:1.0 H except for the uppermost 47 feet which was vertical (refer Figures 10a and 10b) [52]. The modification which also included grouting between the old and new concrete to assure monolithic behavior was completed in 1989. On June 28, 1992, the M=7.4 Landers Earthquake occurred about 28 miles from the strengthened structure. Then one day later, the closer M=6.6 Big Bear Earthquake occurred about 11 miles from the dam on an unnamed fault reportedly in response to the rupture on the Landers Fault. At the Big Bear Lake Civic Center about 2.4 miles from the dam, a strong motion instrument recorded a peak of 0.57g horizontally and 0.21g vertically during the Big Bear Earthquake. This recording is about 5 miles closer to the causative fault than the dam. Thus, the estimated PHGA at the dam was between 0.4 to 0.5g. A thorough inspection after the earthquake indicated that the dam had not been damaged. The only damage was a slight displacement of the highway bridge at the crest of the dam. The bridge has since been removed from the dam and relocated downstream.

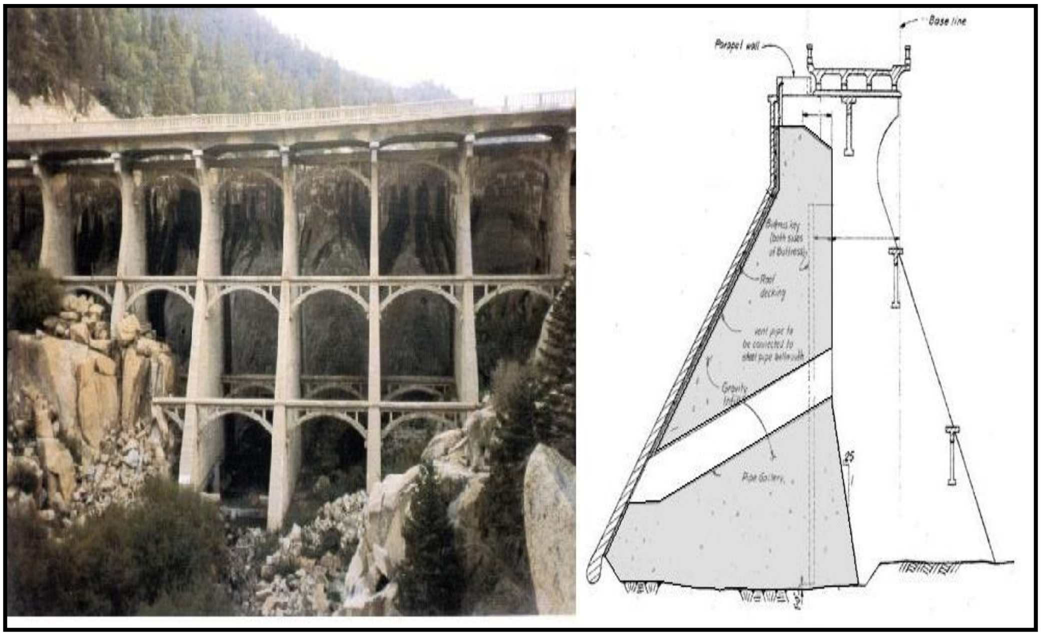

Base line
Parapet wall
Buttress key (both sides of Buttress)
Roof decking
vent pipe to be connected to steel pipe bellmouth
Gravity infill
Pipe Gallery
25
1

Figure 10: Bear Valley Dam (a) multiple arch dam. (b) Shaded area Shows concrete infill between arches to improve seismic stability [52].

2. Stewart Mountain Dam

This Dam is located on the Salt River east of Mesa, Arizona. The concrete structure includes thin arch dam, two thrust blocks, three gravity sections, and one spillway, Figure 11. The arch has a 212 foot (65m) structural height, an 8 foot (2.4m) crest thickness, a 34 foot (10m) base thickness, and a 583 foot (178m) crest length. Construction was completed in March 1930. Concrete placement proceeded without cooling in 5 foot (1.5m) high blocks. Lack of cleaning horizontal construction joints before subsequent placements produced weak layers of laitance at every lift surface, thus resulting in planes with little or no bond. The combination of weak horizontal planes and vertical contraction joints basically produced independent concrete blocks. Alkali-aggregate reaction within the concrete has permanently displaced the arch dam crest 6 inches upstream and 3 inches upward. This reaction has apparently stopped due to depletion of the alkalis in the cement and is no longer a major concern. A structural analysis indicated large inertia forces near the crest that are produced in the 3D linear elastic modal-superposition time-history analysis with mass-less foundation and (Westergaard Added Mass) using the MCE of Richter magnitude M=6.75 at 9.3 miles (15 km) occurring on the Sugarloaf Fault. Predicted horizontal site accelerations are approximately 0.34g upstream/ downstream, 0.258g cross stream, and 0.19g vertical using attenuation curves developed by Seed and Idriss. The study determined at times, that during the design earthquake, the upstream inertia force tended to pull portions of the upper arch dam horizontally apart along the vertical joints. Concrete blocks in these areas may thus be without much side support from the natural arching action of an arch dam. The arch dam was believed not to perform dynamically as a monolithic unit as originally intended, because of the unbonded horizontal lift surfaces. This situation is acceptable for static loadings because of the horizontal arching action transferring loads along the arches and 'wedging' the structure in the canyon. The cantilevers still act as cantilevers because of the transfer of load across the lift surfaces from friction, but this situation was determined to be not acceptable for dynamic loadings. Large inertia forces induced in the upstream direction of the dam during an earthquake were greater than the static reservoir load. It was felt that individual concrete blocks moving in the upstream direction without arching action would break the frictional resistance along unbonded lift surfaces causing instability. A progressive failure of the top central section of the dam was postulated. The dam was modified in 1991 with post-tensioned cables adding the necessary normal force across these horizontal planes to increase the frictional resistance for stability. The cable installation consisted of 62 epoxy-coated post-tensioned cables varying from 75 to 240 feet (23 to 73m) in length, inclined between vertical and 8° 40' off vertical. It is believed that this modification was the first time an arch dam had been post-tensioned [52].

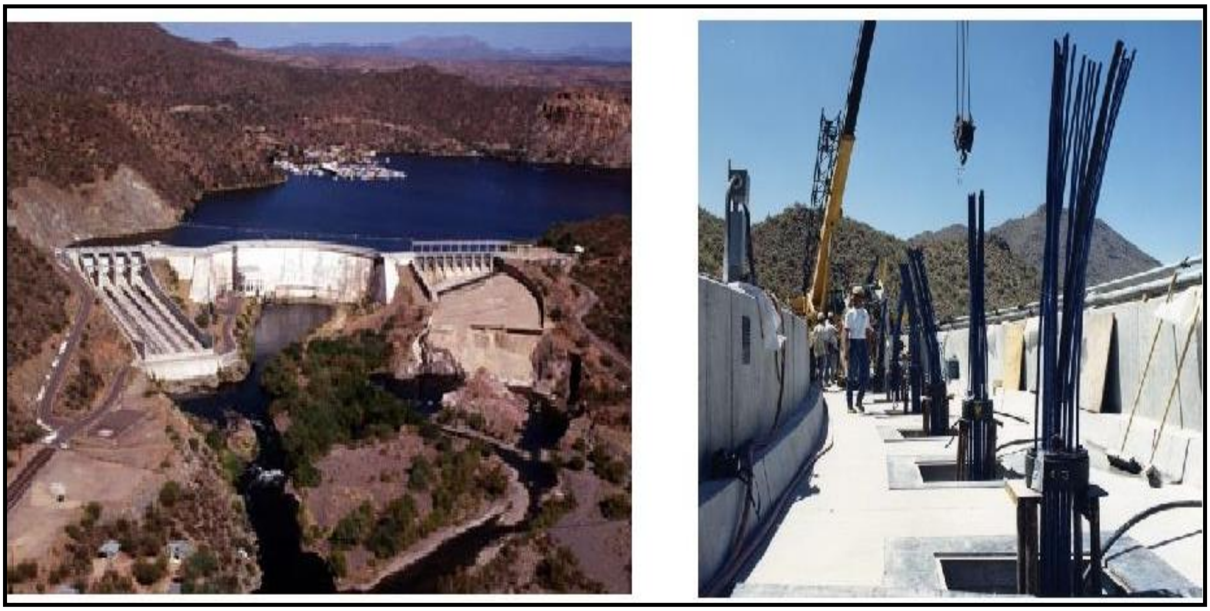


Figure: 11: (a) Stewart Mountain Dam, (b) Post-tensioned cables installed to increase frictional resistant along lift lines [52].

3. Big Tujunga Dam

Big Tujunga Dam was a thin-arch concrete dam completed in 1932 on Big Tujunga Creek as a flood control structure northeast from Los Angeles and is now also used for water supply. It has a height of 244 feet (74m), a crest length of 830 feet (253m). The arch is 8-feet (2.4m) thick at the crest and a maximum base thickness of about 74 feet (23m). It is owned by the Los Angeles County Department of Public Works (LACDPW). The project has required numerous improvement programs, studies, and remedial actions to address seepage and rock stability issues. In 1975, the owner conducted a 3D finite element seismic stability study after the 1971 San Fernando Earthquake had damaged Pacoima Dam about 12 miles away from Big Tujunga Dam. The seismic time-history record for the MCE used in the study was a modified recording measured during M=7.5, 1971 event at Pacoima resulting in a PHGA of 1.1g on the Sierra Madre fault. The conclusion from the study was that Big Tujunga Dam was unable to adequately resist the MCE with a full reservoir. As a result, the preferred interim alternative was to restrict the reservoir to 77 feet below the spillway crest, a level determined safe for the MCE by limiting peak tensile stresses in the dam to below 750lb./in^2 as determined to be the maximum dynamic tensile capacity in 1975 based on core testing. In 1996, MWH Global reanalyzed the seismic performance of the dam for the MCE and again determined the structure inadequate. In 1999, LACDPW secured funding to rehabilitate the dam for both seismic and flood conditions. Of the various concepts to stabilize the dam for a large seismic event, the preferred alternative was to convert the thin-arch to a thick-arch dam by adding a conventional concrete overlay against the downstream face; adding 12-feet thickness to the crest and 66-feet thickness to the base, refer Figure 12.

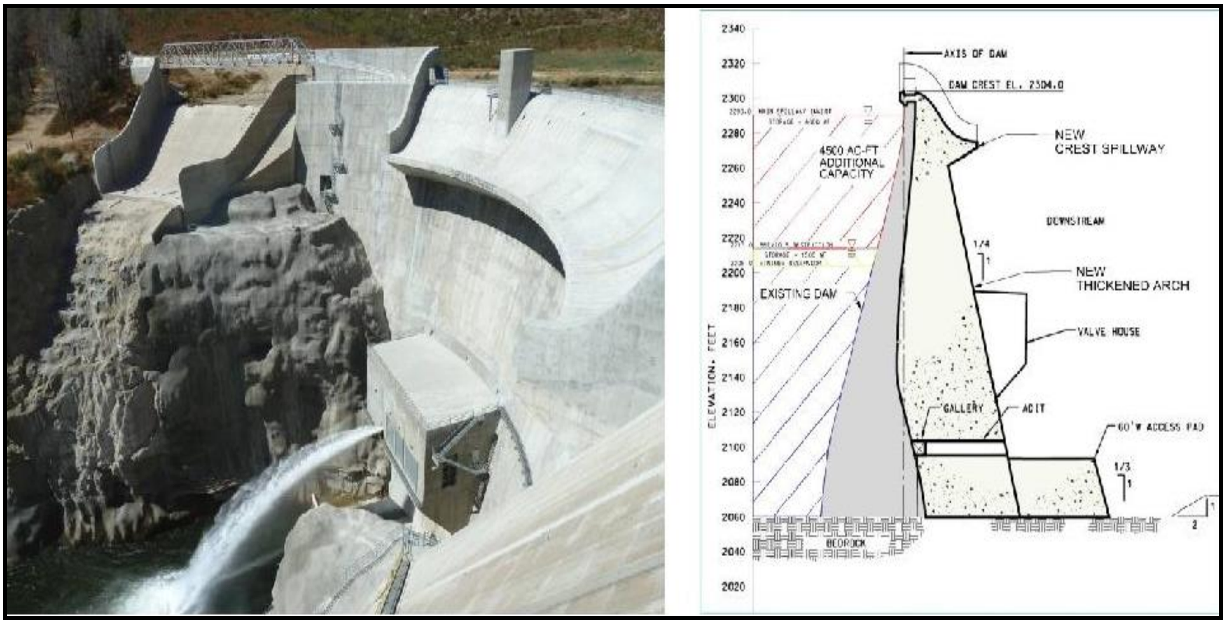


Figure 12: Big Tujunga Dam after modification [52].

The 3D linear-elastic seismic finite element analysis of the modified structure showed limited isolated areas at the upstream face of the dam of high stress during a small number of acceleration time history peaks that exceeded the estimated tensile strength of the concrete and thus not deemed critical. Steel dowels grouted into the existing dam and protruding into the overlay concrete spaced at 10-foot by 10-foot grid pattern provided additional strength, beyond natural cohesion, between the new and overlay concrete. Foundation consolidation grouting on 10-foot centers in each direction reduced the possibility of differential settlement between the existing and new structures.

12. Conclusion

Earthquakes can have a profound impact on dams' safety, especially in seismic active zones. Historical records indicate much dam failure due to earthquake and even more dams, which suffered varying degrees of damage(s). Factors which have control over this are first those related to the nature of the event, i.e. the strength of the earthquake which denotes the amount of energy released and the proximity of the dam from its epicenter or its location from the fault line, and second those factors related to the type of dam, its material and method of construction, the design criteria, and analytical methods used in its design. Most failed dams were tailing dams or hydraulic fill dams or small earth fill dams which reflect the weight of the aforementioned factors. Generally speaking, embankment dams are less tolerant to ground shacking caused by an earthquake than earthquakes. Failure modes or damage type of embankment dam are due to settlement, cracking or in some cases liquefaction. Rockfill dam has shown very good performance even when compared to concrete dams. On the other hand, many concrete dams have sustained various degrees of the damage(s) and only one gravity dam (barrage) had failed, and that was a special case of being located on the line of a fault which underwent considerable vertical and horizontal displacements. As far as damages to concrete dams, buttress dams have shown less tolerance to earthquakes when compared to arch and gravity dams. RCC dams which are a recent development in dam design and construction have shown excellent records although some of them are very large dams and this is attributed to the nature of construction material, and the up to date design criteria used. In modern dam designs, engineers have abandoned the obsolete empirical Pseudo-static method towards a more rational approach known as the dynamic response method which adopts the expected ground shaking expected from anticipated earthquakes based on actual seismic records of events in the region where the dam is located. This has helped considerably to build very large dams in such a region with more confidence in their safety, whether concrete or earth fill dams. In all cases, however, the designers of dams are called upon to look into the many hazards, such as seiches and landslides that may still exist and work to reduce their impacts on the safety of the dam. Good observation of dam behavior during and after the earthquake, by visual inspection and examining seismic records helps considerably in evaluating the dam safety and decide on corrective measures; if needed. It follows; therefore, that installing seismic instrumentation in dams is very important and should be provided. Every dam that is built represents a unique case as far as its reaction towards earthquakes, especially in view of the different ways that such earthquake can happen; it follows that studying each dam case gives additional lessons to be added and enrich knowledge in this field and help in design safer dams.

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Dams Safety and Geology

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Abstract

Geological hazards that can face dams are very important in deciding their safety and successful performance during their lifetime without excessive and costly repairs. Recognizing such hazards must be made at an early stage of the investigation works. Geological hazards which have caused dam failures or resulted in redundant reservoirs can vary between presence of karsts in the reservoir or in dam foundation, presence of soluble rocks, hidden faults, or the presence of hazardous materials. Learning from case histories of dam failures and incidents is important to avoid problems raised by these hazards.

Many such cases are presented in the preceding paragraphs to show the variety of such problems and help understand their nature. These case histories, can help the designer in the selection of the most appropriate type of dam suited for a particular geological condition, avoiding dangerous situations such as but not limited to excessive or differential settlement. Understanding the real conditions of foundation decides also the efficiency, scope and type of foundation treatment. The Teton dam failure given in this paper illustrates one case when such understanding was missing. In any case, the active participation of geologists working with the designers in all stages of dam construction process is very important to eliminate or reduce to safe limits any geological hazard that can the future dam.

Keywords: Geological Hazards, Geological Investigations, Karsts, Faults, Soluble Rock, Foundation Treatment.

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1. Selection of Safe Dam sites

The most important elements in dams’ sites selection are topography and geology. While topography defines the size of the dam; its height, length and volume of reservoir, geology, on the other hand dictates the possibility of building safe dam with respect to tightness of the reservoir, stability of the reservoir slopes, in addition to the foundation conditions of the dam itself. Economic and financial factors play their role in site selection when figuring the amounts and costs of foundation treatment required for a safe and reliable dam.

It is important to emphasize here that no restrictions should be put on the magnitude of geological investigation required for site selection and for sound planning and design; as far as the cost is concerned. This cost should be commensurate to the importance of the dam, and the safety hazards that it can create to society. Moreover, this cost is normally payed back by choosing a more economical design with enough level of safety to answer for the worries related to loss of human life and other material losses as expressed by the society and its institutions, in addition to trouble free performance, which will, at the same time, cuts on the cost of operation and maintenance of the dam.

It is not intended here to define the detailed requirements for the geological investigations necessary for selecting a good and safe dam site free from risks, as this must be done case by case, but a general outline only may be given.

For preliminary studies, the dam site investigations shall concentrate on selecting the dam axis and the locations of the appurtenant structures considering the best topographical and geological features by utilizing geophysical surveys, drilling boreholes and excavating some test pits and trenches or even investigation galleries if required. Sampling and laboratory testing of soil and rock samples then follow. At this stage, inspection of the reservoir area is warranted to check for salient features such as potential landslides, which may threat the future dam safety, and sinkholes or karst that reduces reservoir tightness or even endangers the stability of the dam. Failure to do this may lead to catastrophes like the Vajont Dam land slide which caused the death of thousands of people, or may lead to creating a redundant dam if the reservoir water leaks out unhindered as the cases of Hondo Reservoir in New Mexico, Cedar Canyon Dam in South Dakota and Anchor Dam in Wyoming (USA). Anchor Dam was built despite more than 50 sinkholes and stream losses associated with karstified Triassic gypsum that was reported for the reservoir area. During the first attempt to fill the reservoir, numerous additional sinkholes were reported. A collapse around 100m across and 20m deep occurred upstream of the concrete dam and was isolated by the construction of an additional dike that limited the operational water level of the reservoir and reduced by 50% its storage capacity. After 30 years of costly attempts to repair the reservoir, it has never held more than a small pond [1]. For large dams, where safety must be reasonably ensured beyond any doubt, the next stage of the geological investigations shall be detailed and must concentrate on getting accurate description of the foundation stratigraphy and structural setting by getting full geological profile along the dam axis which should extend to a safe distance in the abutments. Moreover, geological cross sections at selected points along this axis and at appurtenant structures locations must be prepared. All this shall be accomplished by driving deep boreholes extending down to depths (deeper than the foundations) sufficient to catch any possible geological anomaly. In situ permeability test may also need to be carried out in these boreholes to give a representative permeability picture at the various depths under the dam, while soil and rock samples shall be obtained from these boreholes for laboratory testing to investigate the type of rock, their properties and strength characteristics. Moreover, these boreholes should be geophysically logged to indicate any ambiguous data acquired from geological mapping. Design engineers and field geologists must be well versed in the geological hazards that threaten the safety of dams and should be capable of interpreting the geological finding correctly. According to any special situation, these may include among other things appreciation of regional earthquake hazards and dangers inherent in the proximity of dam sites to potential or actual active faults. Other important factors shall be studied carefully and not overlooked such as; potential landslides, the presence of karsts and soluble rocks, foundations containing compressible layers, expansive soils or weathered rock, existence of faulting under the dam and their orientation, presence of weak sliding surfaces such as thin seams of clay, etc. The extent of foundation treatments required to attain the prescribed safety level shall be decided on the light of these studies with the aim of eliminating all potential geological hazards.

2. Land Slides Hazards

The subject of landslides has been treated in some details in a previous paper entitled “Dam`s safety and Earthquakes” written by the authors; it deals with earthquakes as the prime cause of landslides of mountain slopes into reservoirs and the hazards they may create to dams, so the reader is invited to refer to it as a complementary reading.

Many dams and reservoirs are constructed in steep mountainous terrain where landslides can occur. Landslides, if large enough, can affect the safety of a dam or reservoir if they fail or move. Landslides generally are triggered by the saturation of soil mass, which may slide down in shear failure along some weak layer within the geological beddings within the slope. The chances of such failure are intensified by wetting of the mass from excessive rainfall, snowmelt, or large earthquakes, also from the quick draw down of the water level and are called “reservoir induced landslide”. Landslides are additionally affected by anthropogenic factors associated with the filling or operation of the reservoir, e.g. morphology of the flooded slope, road cuttings on the reservoir rim, changing in the course of shoreline or considerable rapid reduction of water level and such sudden changes that affect hydrostatic groundwater pressure in the slope. These factors often coincide leading to a sudden drastic decrease in slope stability increasing the activity of landslide. The rise in pore water pressure on the slope results in changes in geotechnical parameters of the soil, mostly reducing its cohesion and shear strength and increasing its weight and bulk density [2]. Old landslides can be reactivated, if the geologic conditions are adverse, new landslides can be triggered. New landslides develop in most cases after some recent activities, which drive the soil or rock mass gradually to a state of limiting equilibrium, such as impoundment of the reservoir, excavation of a new road cut, or other disturbances to the area. Landslides can occur upstream in the reservoir, in a canyon downstream of a dam, or even within the abutment of a dam.

An idealized form of landslides is shown in Figure 1 with the commonly used nomenclature [3].

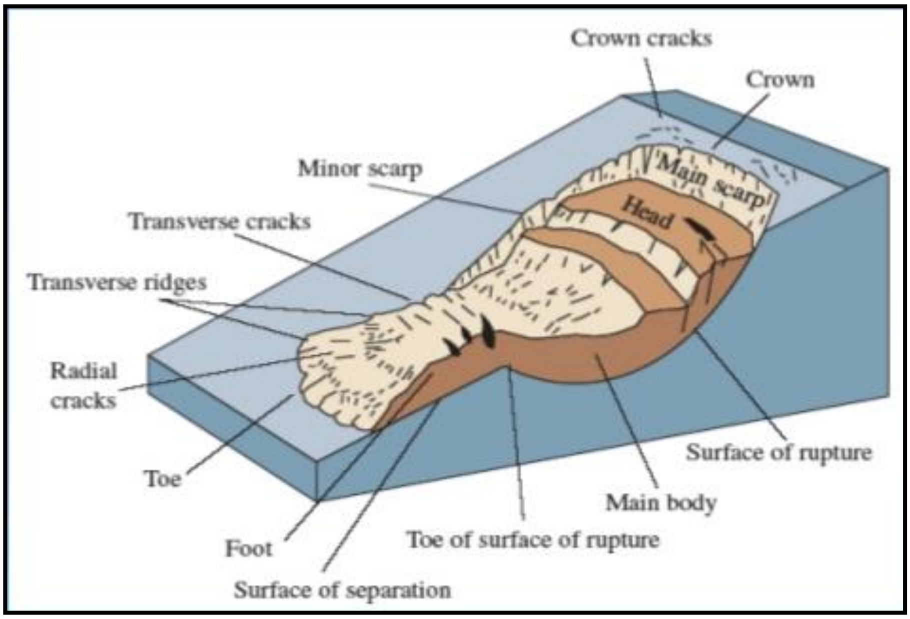


Figure 1: An idealized landslide and commonly used nomenclature [3].

Landslides occurrence can take many scenarios: The first one is when such landslide falls in the reservoir of an existing dam, the second is when it occurs downstream from an existing dam, and the third is when a new dam is built abutting an old landslide which starts moving after construction of the dam. For the first scenario, if a landslide is failing into the reservoir, and if it is close enough, large enough, and moving fast enough, it can generate a wave large enough to overtop a dam. Sloshing back and forth in the reservoir can result in multiple waves overtopping the dam. If the waves are large enough, downstream consequences can result just from the downstream overtopping flows even if the dam does not fail. If enough large waves overtop an embankment dam or a concrete dam with erodible abutments, an erosion failure could potentially result. One catastrophic case of landslides in conjunction with dam reservoir is the Vajont Dam reservoir landslide. This event occurred on October 9, 1963, at 10:39 pm when 260 million cubic meters of rock broke off from the top of Monte Toc, on the border between Veneto and Friuli Venezia Giulia. It fell into the reservoir of the Vajont Dam, producing an enormous wave of at least 50 million cubic meters of water. The dam completed in 1959 and one of the biggest in the world at the time did not suffer any serious damage. However, flooding destroyed several villages in the valley and killed almost 2,000 people. A third of the population of Longarone, the largest village downstream from the dam, perished [4], refer Figure 2. Post-failure investigations showed that low strength clay layers existed between the limestone beds. It was surmised that rainfall on the mountains above the reservoir was conveyed through solution features to the reservoir slopes where it became trapped by the impermeable clay layers. A review of the landslide survey data showed that movement accelerated with a high reservoir level following periods of heavy rain. Thus, it was the combination of high reservoir level, which unweighted the toe of the slide, and heavy precipitation which increased the pore pressures in the rocks of the slope, that lead to triggering of the slide on weak clay layers. This was confirmed by limit equilibrium analyses that considered side constraint afforded by a fault on the upstream side of the landslide mass [5] and [6].

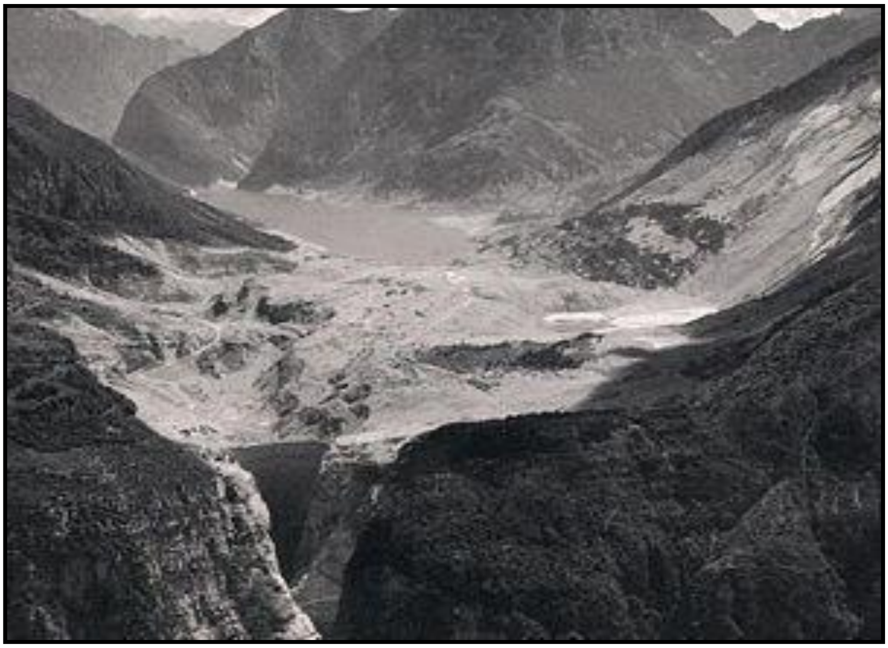


Figure 2: View of the Vajont reservoir after the landslide (1963) [4].

In the second scenario, when a landslide occurs downstream of a dam, it can then block the river course creating a landslide or debris dam. The water impounded by a landslide dam can create a dam reservoir (Lake) that may last from short times to several thousand years, and because of their rather loose nature and absence of controlled spillway, landslide dams frequently fail catastrophically and lead to downstream flooding, often with high casualties. A common failure scenario is overtopping with subsequent dam breach and erosion by the overflow stream. Operational releases from an existing upstream dam can also overtop and erode the debris dam, sending a large slug of potentially life-threatening flows downstream. In such case, releases from the upstream dam must be controlled with caution, which may limit these releases and consequently, limit the benefits derived from the operation of the dam such as power generation or irrigation. While damages can occur whether there is a dam upstream of the landslide or not, having an upstream dam creates added complications. In an emergency case threatening the safety of the upstream dam, say from large earthquake ground shaking or any other reason, it may not be possible to lower the reservoir for fear of overtopping the debris dam. In flood control operations and when necessity requires high releases from the dam, these releases can cause overtopping erosion failure of the debris dam and a sudden surge of water downstream. For this reason, there may be pressure on the dam operators to hold back the downstream flows. If the flows cannot be held back, it will likely become the task of the dam operators to issue warnings. For all these reasons, it is necessary to have a fair idea and good understand of the geology in the canyon downstream at an early stage during the investigation and planning of the dam and to recognize the potential development of this type of l failure mode in the future. There is also the possibility that high-water levels created upstream from the landslide dam may impound the downstream facilities of an upstream dam causing damages and might even take out these facilities out of operation. Giving some examples of the occurrence of some landslide’s dams may be done to highlight the magnitude of the impacts of such dams on river courses and the extent of dangers they pose to existing dams if they are close enough to such dams. One such example in modern history is Usoi Landslide Dam in Tajikistan created by a landslide triggered by an earthquake on February 18, 1911. It dammed the Murgab River to the height of 570m (1,860ft) to impound Sarez Lake 505m (1,657ft) deep. The second case is the Lake Waikaremoana in New Zealand which was formed by a 250m (820ft) high landslide dam believed to be 2,200 years old. Between 1935 and 1950, the landslide was tunneled and sealed to stabilize it so it could be used for hydroelectric power generation. This appears to be the first example of modification of a natural landslide dam for power generation. Among the most destructive landslide lake outburst floods in recorded history is the one that had occurred in the Sichuan province, China, on 10 June 1786 when the dam on the Dadu River burst, causing a flood that extended 1,400km (870 miles) downstream and killed 100,000 people [7].

In some cases, dams have been built abutting against a landslide. Often, these are ancient landslides that have stopped moving, or are moving very slowly. However, if such a landslide moves far enough, it can crack the core of an embankment dam, resulting in pathways for internal erosion to initiate, or disrupting the abutment support of a concrete dam, resulting in cracking and structural collapse of the concrete. If a thin upstream member, such as a reinforced concrete facing, forms the water barrier for an embankment dam, landslide movement can lead to buckling of the slab, and flow through large cracks, which may be capable of eroding even large rockfill forming the downstream shell. If the abutment of an embankment dam contains a landslide, limit equilibrium analyses, including possibly reliability analysis, can be made to estimate the likelihood of the landslide moving under various loading conditions (i.e. earthquake loads ranges, flood loads ranges, and/or ground water ranges). These load ranges can be put on the front end of an event tree and the likelihood of cracking estimated for each. If cracking is recognized as a possibility, then all measures to limit internal erosion must be considered to decrease the risk of failure. It must be emphasized, however, that sites with potential landslides shall be avoided altogether by carrying good and intensive investigation, whether by drilling and geophysical surveys, at the proposed site.

If a landslide, very close to the upstream face of the dam, is triggered by a large rainstorm, then damage to a spillway or other outlet features can occur at the same time when large reservoir inflows are occurring. If the landslide debris block the spillway or intakes or damages them to the point that they cannot be operated, then premature overtopping of the dam may occur. In this respect the case Derbendikhan Dam landslide in Iraq may be given as an example of such potentially dangerous landslide and what actions may be taken to stabilize it.

Derbendikhan Dam is located on the Diyala river approximately 65km south-east of Sulaymaniyah and 230km north-east of Baghdad. The scheme is a multi-purpose hydro-complex and the main structure is a 128m high embankment dam with a central clay core and rockfill shoulders. The crest length of the dam is 445m. The dam is constructed in a narrow steep-sided gorge on a series of sedimentary rocks including, marls, sandstones, limestones and conglomerate.

The reservoir impounded by the dam had a total design capacity at the normal operating level (El. 485.00m a. s. l) of 3,000 Mm^3, of which 2,500Mm^3 is live storage and 500Mm^3 being dead storage. The current storage volumes will be less than this due to almost 60 years of sedimentation. The reservoir is controlled by a gated spillway structure located on the right bank and comprises three tainter gates installed on an ogee shaped overflow section followed by a steep chute terminating in a deflector bucket. Dividing walls separate the discharges from each of the three gate openings. The maximum discharge capacity of the spillway at the normal operating level (El. 485.00m a.s.l) is 5,700m^3/s and 11,400m^3/s at design flood level (El. 493.50m a. s. l).There are two low-level outlet tunnels (6m and 9m diameter) connected to steel conduits, which discharge water through three irrigation valves. The conduits also originally supplied water to two small turbines for the service generating station and are now connected to the three large turbines of the new powerhouse. An intake tower for the irrigation outlets is located between the spillway approach channel and dam embankment. The intake tower houses three vertical lift gates, gate No. 1 serves the 6 m diameter tunnel; gates No. 2 and No. 3 serve the 9m diameter tunnel. Construction of the dam started in 1956 and was completed in summer 1961. Five years after first impounding of the reservoir, slip failure of an area of the right bank some 300m upstream of Derbendikhan Dam was identified in November 1967. The affected area, consisting of about 2Mm^3 of material approximately 350m in length and 200m wide, moved 1.5m downwards at reservoir elevation 455m following 126mm of rainfall over a period of 7 days, refer Figure 3.

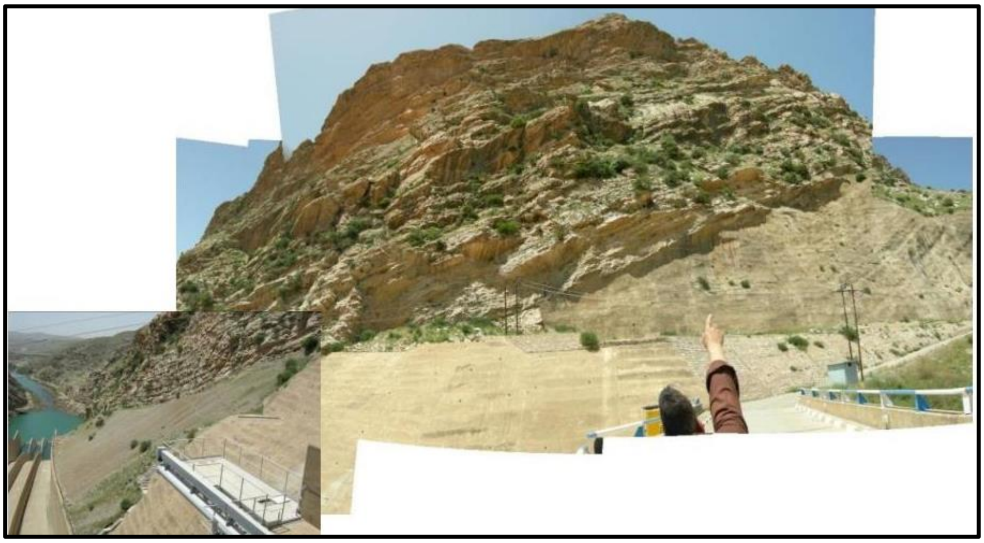


Figure 3: Rock cliff above right abutment overlooking spillway chute at left, and landslide area at right lower part of the picture.

Additional settlement of the right bank slip area occurred between November 1969 and January 1970. A further failure of this slip mass represents a threat to the safety of Derbendikhan Dam as it could potentially cause the following:

a. Blocks the area immediately in front of the inlets and the spillway, restricting the volume of water that could be safely passed through the irrigation outlets, hydropower tunnel intake and over the spillway in the event of an extreme flood
b. Water displaced by the slip mass could cause a flood wave to overtop the dam crest, if the slip failure occurred at its highest reservoir level.

It is therefore, extremely important that a long-term solution for the stabilization of the right bank is achieved in the interests of dam safety.

Some remedial works for the stabilization of the slip area were proposed by a French Consultant consisted of:

a. Unloading of the slip area by excavation and removal of slip material above elevation 488m a.s.l.
b. Surface protection of the slip area by a drainage system and by landscaping to minimize the infiltration of rainwater
c. Construction of 9 wells with horizontal drains at the bottom, equipped with pumps to lower the phreatic surface within the sliding zone (deep drainage)
d. Restriction of the reservoir drawdown to a minimum level of 460m.

These remedial works were commenced in 1979, but they remain incomplete as they were halted in October 1980 due to the Iran-Iraq war affecting activities at the site. The status of the remedial work at that time was reported to be:

a. Excavation of 120,000m³ above elevation 488m nearly completed, but not assessed by topographic survey.
b. Placement of topsoil for landscaping not started.
c. Excavation of 7 wells were started; well P-2 collapsed and was replaced by well P-56; wells P-3 and P-7 were completed including the sub-horizontal drains.
d. Installation of 12 piezometers completed.

In January 1981, complementary stability analyses by the same consultant indicated that the minimum reservoir level could be revised and lowered to 455m.

The dam inspection carried out by a second consultant in January 1998 suggested that no significant movement of the right bank slip area had occurred since 1986, despite the drawdown of the reservoir to level 442m in 1988. New cracks on the ground were observed in April 1998, possibly indicating further movement of the slip mass, but the development of these cracks could also be partially attributed to surface run-off erosion during several days of high intensity rainfall.

In September 1999, the reservoir level was once more drawn down below the recommended safe level of 455m and the first consultant was engaged again to assess the risk of movements. The following recommendations were given in December 1999:

a. Installation of pumps in the completed wells and start draining the wells.
b. Cleaning of the sub-horizontal wells.
c. Collection of rainfall runoff by installing surface drainage.
d. Cleaning of existing piezometers.
e. Survey of all existing beacons to monitor movements.
f. Underwater survey of the concrete slabs installed at the toe of the slip slope (these were last surveyed in November 1978).
g. Improve/complete the landscaping of the slip area.

The status of the wells installed in the right bank slip area was followed and summarized, and analysis was carried out by the consultants in 2006 to examine the behavior of the slip area. Their report makes further recommendations for the continued monitoring of the slip area and short-term measures.

The main recommendations can be summarized by:

a. The allowable minimum drawdown level of the reservoir may be reduced to elevation 450m, provided that a maximum drawdown rate of 10cm/day is not exceeded.
b. Urgent attention to reinstating/establishing an effective network of surface drains to intercept surface run-off and protect the slip area from surface run-off erosion.
c. Urgent attention to the cleaning of the sub-horizontal drains in existing wells to maximize their drainage efficiency.
d. Continues monitoring installed drainage wells collection of pumping data to assess the performance of the existing well installations.

The conclusions of the report were that, in order to achieve the minimum drawdown level of 435m (original design) and restore the full operating capacity of the reservoir, the current data suggests that the sub-horizontal drains in wells P-1, P-6 and P56 should be installed, a seventh well (P-15) installed and the surface drainage/landscaping of the slip area completed. The report also suggests a two-stage approach towards the realization of the long-term aim of the remedial works (i.e. a safe minimum drawdown level of 435m):

a. Immediate (short-term) implementation of the main recommendations indicated above.
b. Reassessment of the slip area behavior after a further period of monitoring to determine whether all or part of the outstanding remedial works per the original remedial works design) is necessary to secure satisfactory conditions within the slip area for the safe drawdown of the reservoir level to 435m.

There is still, however, no firm evidence, which confirms that the proposed deep drainage system will achieve its final objective and keep the phreatic surface in the slip mass below or at the limits required for safe conditions with the reservoir drawdown to its intended minimum operating level of 435m. The 2006 analysis yields factors of safety against a deep sliding failure (down to elevation 405 m) of only Fs=1.13 to 1.14 (reservoir level 455m) and Fs=1.10 to 1.11 (reservoir level 450m). These results are below the normally accepted minimum factors of safety required by international standards for load cases without earthquake load. There is therefore, a real concern that the proposed deep drainage system will not be able to achieve its objective and satisfy international standards for slope stability. Furthermore, the design concept of the deep drainage system relies on the continuous reduction of the water table in the right bank area by pumping water from the well installations. This will require maintaining a continuous power supply to the pumps, ensuring that the pumps are regularly maintained, and adequate spare parts are readily available; failure to keep up the required level of maintenance will result in an increased risk to the safety of Derbendikhan Dam. Whilst alternatives for a permanent solution may require a greater capital investment (since significant investment in the deep drainage system has already been made), the ongoing maintenance costs of the deep drainage system may also mean that an alternative solution with no associated maintenance costs could be economically advantageous. A no or low ongoing maintenance’ solution would also present a lower risk to the safety of Derbendikhan Dam.

A common solution for the long-term stabilization of a slip area is to load the toe of the slope. Given the proximity of the slip area to the power intake structure, irrigation outlets and spillway, consideration may be given to the placement of interlocking pre-cast concrete blocks to load the toe area. Whatever the design of the permanent solution, the impact that any proposed remedial works will have on the performance of the intakes, and the spillway needs to be evaluated and properly accounted for [8].

In Summary, the actions proposed and taken to increase the stability of the sliding mass and reduce the possibility of complete failure were to reduce the weight of the sliding mass by removing sizable part of it from the top, lower down the phreatic surface by pumping seepage water from deep drainage well and to bring this surface below the slip surface, and finally to reduce infiltration of rainwater into the mass which raises the phreatic surface by collecting rain water by good surface drainage and remove it from the slip area, and finally by limiting the drawdown the reservoir as a higher water level offers a stabilizing force exerted on the mass and increases its stability. Such work requires continuous operation of the pumps, in addition to continuous revision of the stability and the operation rules of the reservoir. Limiting the drawdown of the reservoir water level and not being able to reach the designed lower operation water level represents a loss of valuable storage capacity and all the benefits associated with it, while at the same time reaching this level poses a real threat by accelerating the movement of the slip mass.

The presented solution and performed works are of doubtful nature and requires constant vigilance and high maintenance costs, which emphasizes the need for a permanent solution, especially as the dam is located in an active seismic area which adds to the instability condition of the slip area. This safety problem is exasperated by the latest 7.3 magnitude seismic event the dam was subjected to, which occurred on 12th of November 2017. This earthquake struck the Iraq-Iran border with the epicenter in the Iranian city of Sarpol-e Zahab whereas Derbendikhan was the most impacted city in Iraq. The earthquake was one of the largest earthquakes occurring in the Iran–Zagros Zone since 1900. About 396 lives were lost, and 7000 people were injured on both sides of Iraq–Iran border, with most fatalities occurring in the city of Sarpol-e Zahab, whereas Derbendikhan was the most impacted city in Iraq. The earthquake struck the northeast of Iraq, with its epicenter 30km away from the Derbendikhan dam, and there is now a serious concern about the Derbendikhan dam’s and the landslide safety.

The dam was previously evaluated to be seismically safe under a maximum probable shaking of Mw 6.5, and no study is done for the seismic stability of the landslide. Furthermore, it was reported, however, that there was further concern about the landslide in the right bank upstream of the dam and rock falls from the high cliff in the left bank. The deformation of the dam body after the earthquake is visually apparent, and the State Commission on the Survey of Iraq observed several fissures on the crest soon afterwards. In the following months, this region exhibited 53 aftershocks with Mw > 4 [9]. A preliminary inspection was carried out by a team from the Ministry of Water Resources, which documented the cracks in the dam body and one new crack in the area above the limits of the landslide shown in Figure 4, but it seems that this team did not check new cracking in the landslide area which is located below the access road to the dam [10].

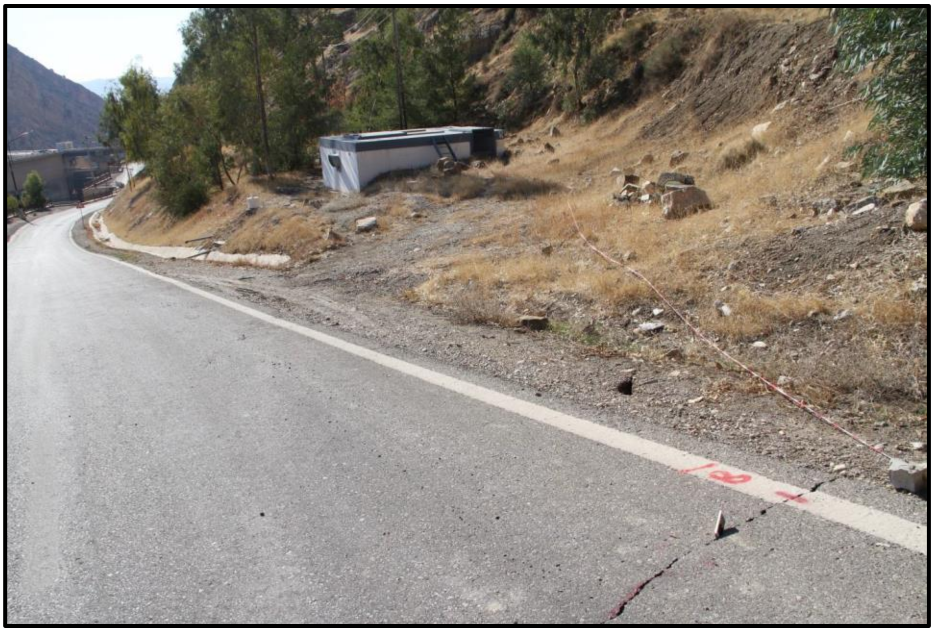


Figure 4: Location of a crack along the area above the landslide area (Crack is marked by red and white rope) and then extends down across the paved road down into the landslide area.

In a paper entitled “Global Losses from Land Slides Associated with Dams and Reservoirs” it is stated also that the risk of landslides can be exasperated by the seismic activity resulting from impounding large reservoirs known as Reservoir Induced Earthquakes (RTS). From a technical perspective, it may be said that seismic activity can profoundly alter the rates of activity of the landscape, such as that conditions that apply during a site investigation phase may no longer be current [11]. It seems that no meaningful technical evaluation of the dam and the landslide area has been carried out so far, which reflects the state of negligence of a government which is sunk in corruption and political bickering not giving heed to a safety risk which threatens millions of the population of Iraq. The consequences of complete failure of this mass are grave as already stated. It should be remembered that the breaching of the dam in case of overtopping may lead to a flood wave in the downstream of the river and the possible failure of Hemren dam, which is located further down on the Diyala rivers. This can lead to national catastrophe by flooding very large area, including the capitol Baghdad. If a landslide exists in a concrete dam foundation, movement can also cause cracking of the dam. Similar methods to those summarized above for embankment dams can be used to evaluate the potential for movement, except that internal erosion is not the issue, but adverse cracking, isolation, and displacement/rotation of the isolated blocks of concrete in the dam could be. See the section on Risk Analysis for Concrete Arch Dams for some ideas on how to evaluate the risks associated with this. If movement of the landslide results in loss of abutment support, such that the dam moves with the landslide, the methods in the section on Risk Analysis for Concrete Arch Dams to sliding of foundation blocks can be used to evaluate abutment stability and potential risks.

3. Karst and Soluble Rocks Hazard

Karst is a topography formed from the dissolution of soluble rocks such as limestone, dolomite, and gypsum. It is characterized by underground drainage systems with sinkholes and caves, refer Figure 5 [12]. It has also been documented for more weathering-resistant rocks, such as quartzite, given the right conditions. Subterranean drainage may limit surface water, with few to no rivers or lakes. However, in regions where the dissolved bedrock is covered (perhaps by debris) or confined by one or more superimposed non-soluble rock strata, distinctive karst features may occur only at subsurface levels and can be totally missing above ground.

Dams are costly structures whose failure can lead to disaster, large scale mortality and financial liability. Water leakage from reservoirs through ponors, sinkholes and karst conduits can lead to costly inefficiency, or even project abandonment. The unnaturally high hydraulic gradients induced by the impounded water may flush out of the sediment that blocks karst conduits. It can also produce the rapid dissolutional enlargement of discontinuities, which can quickly reach break-through dimensions with the turbulent flow. These processes may significantly increase the hydraulic permeability of the dam foundation on an engineering time scale leading to instability and failure of the dam.

The main reason of limestone karstification is the dissolving of the limestone by carbonic acid, which is formed by reaction of rainwater with carbon dioxide from the atmosphere. However, the formation of gypsum karstification is accomplished by reaction of oxygen (O_2) in surface water, which seeps into the ground and reacts with sulfide in the system forming sulfuric acid, which then reacts with calcium carbonate causing increasing erosion within the limestone beds. The abundant presences of carbonate sedimentary rocks all over the world and the existence of ambient conditions for the formation of karst has resulted in the existence of extensive karsts areas. But the intensive use of water resources during the last century resulting from the great expansion of economic activities dictated extensive utilization of water resources for irrigation, power and drinking water, which in turns has increased the need for constructing dams and other water related engineering works. Building of more and more large dams and reservoir in karst areas has brought up with it many engineering problems related to karst hazards, which require special attention and understanding.

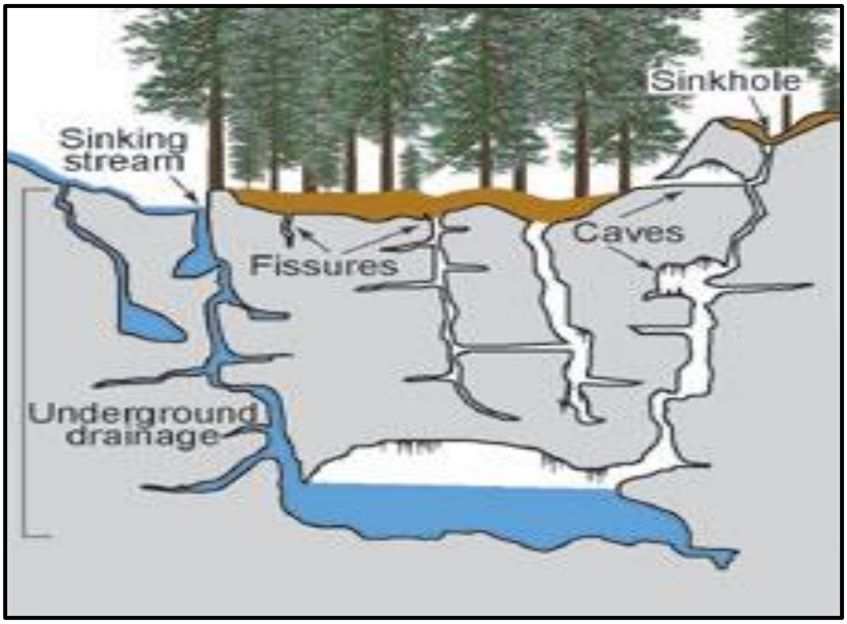


Figure 5: Karst features characterized by sinkholes, caves, and underground drainage systems [12].

Generally, seepage through the foundations and abutments of dams containing soluble rocks may produce settlements and redistribution of pore pressures, which could threaten stability or cause leakage and waste of water sufficient to render the dam uneconomic. If joints or fissures exist in the soluble rock at dam sites, then various kinds of preventive or remedial measures may be considered to provide satisfactory and economic designs. It may, however, be demonstrated by thorough technical and economic studies that a chosen site is unsuitable. If such joints or fissures in the soluble rock abutments or foundations carry sufficient seepage flow of aggressive water, they can enlarge, and flow rates can increase unacceptably. One preventive method is to grout the joints or fissures and thereby seal the flow passages subject to the findings of such studies.

Such hazards; as excessive settlement and/or formation of sinkholes close or under the dam have to be securitized and carefully investigated. A suitable preventive measure might be to construct a groundwater cut- off “upstream” through the soluble rocks. It may be concluded, therefore, that engineering structures founded on or in soluble rocks are at risk if water moving through them is unsaturated in respect to soluble rocks. It is important to anticipate such a possibility during the geological investigation and planning stage and recruits the help of hydrogeologists to evaluate the situation correctly, and if the construction of the dam is to go ahead, then proper protection measures shall be included in the design [13]. The construction of dams and reservoirs in gypsum karst areas locally increases hydraulic gradients and raises a water table in previously unsaturated rocks. Both effects can lead to enhanced dissolution if gypsum rocks occurring at the foundation of a dam construction or within the zone of influence of a reservoir. This can lead to leakage from a reservoir and cause collapses affecting dams and/or the areas surrounding reservoirs. As dissolution of gypsum is much faster than that of limestone, these problems are potentially more severe in gypsum karst than in carbonate karst terrains.

There are numerous examples of dam failure and reservoir leakage due to accelerated development of gypsum karst in many countries. The most infamous failure associated with gypsum was in California, USA, where the St. Francis Dam failed in 1928, at the cost of more than 400 lives and millions of dollars in damages. The problem was associated with dissolution of gypsum that was cementing and filling fissures in gypsiferous conglomerate in the dam foundations. Among other, example is the Quail Creek Dam, Utah, constructed in 1984 failed in 1989, the underlying cause being the dissolution of gypsum in the foundations. Less serious than complete failure, the construction of dams on gypsum (and limestone) karst can lead to serious leakage and inefficiency. This reduces the cost-effectiveness of the structures and in some cases has caused complete abandonment of the project. Numerous dams in the USA either have gypsum dissolution problems or encountered gypsum problems during construction. They include the San Fernando, Dry Canyon, Buena Vista dams. Of special interest is the Hondo and Macmillan dams in New Mexico, USA, as they are associated with gypsum and limestone in which very large dissolutional cavities formed. The proposed reservoirs behind the Hondo and Macmillan dams were never impounded, as leakage was too rapid. In the latter case, huge underground dissolution channels with a capacity estimated at 50 million m^3 were been reported. Emplacement of a cement-grouted cut-off in the foundations of the Red Rock Dam in Iowa, USA, required the injection of about 2800 metric tons of cement into boreholes that intercepted underlying dissolution conduits. The Huoshiro reservoir in China, with a capacity of 4.7 million m^3, was built on a gypsum-limestone karst in Guizhou Province. After a period of water loss, the reservoir eventually emptied through underground routes that connected a series of sinkholes in the reservoir floor to a resurgence point 400m downstream of the dam, where up to 237l/s of water had been discharging. Cases of emptying of smaller reservoirs impounded above gypsiferous formations are numerous in the USA, Western Ukraine, Russia, Siberia, China and other countries. The effect of direct river water action on gypsum outcrops, and of a water table being raised within a gypsum sequence due to dam construction is exemplified by the Kama reservoir, created on the Kama river in the Pre-Urals, Russia, where the river flows entrenched into a gypsum sequence. A rise in water level of 10-15m upstream of the dam, caused a related elevation of the water table within the gypsum of surrounding areas. Gypsum walls along the shorelines were directly exposed to the dissolving action of the reservoir water. The widening of open fissures in the gypsum outcrops during navigation season was measured, depending upon their orientation relative to flow, to vary from 0.2-0.4m to 2.5-3.0m. Five year long observations of particular gypsum boulders submerged in a small gulf, have shown dissolution rates varying from 0.5 to 1.2kg/day/m^2 [14],

[15]. Gypsum ($CaSO_4 \cdot 2H_2O$) may occur in the following forms: 1) as massive beds less than one meter to hundreds of meter thick as parent rock or as thin layers or veins within other rock types, such as shales, siltstones, and carbonates; 2) in a hydrated form as anhydrites ($CaSO_4$) and 3) as disseminated crystals or cement in other rocks. Regardless of the form of gypsum, if present, it typically is the most soluble rock or mineral at dam sites. Therefore, unsaturated water can readily attack and dissolve the gypsum and open up karstic passageways to the flow of water. Karst processes can create and or enlarge vugs, joints, sinkholes, or caverns, and also can produce brecciated rock as a result of gypsum dissolution and collapse of overlying rock(s). Once a through-flowing passage is created in the gypsum, enlargement of the opening results from further dissolution and from abrasion, as water-born particles are transported through the cavity. These karstic openings in the rock may stay open, or subsequently may be filled with clay or other sediments deposited from flowing through waters (refer Figure 6A). Such “clay- filled cavities” can be reopened and reactivated as karsts features if water finds a pathway through and erodes the clay filling. Or if the hydrostatic head forces an opening through the clay filling [16]. A site 30km east of Mosul Dam. Clay filled karst in gypsum (Figure 6B). The holes in the floor of the site (a nuclear site called Atshan) are sinkholes which were not visible on surface (1987). The case of Mosul Dam, Iraq, is an extreme case of karst problems and the grave risks it poses. The dam is an earthfall dam, 113m high, 3600m long, which impounds a reservoir of 11.11 Billion m^3 of water. It was completed in 1986 after 36 years of investigations and studies. The design, however failed to answer to the required level of safety required for such a large dam which threatens, in case of failure, well over one million people (refer Figure 7 and Figure 8).

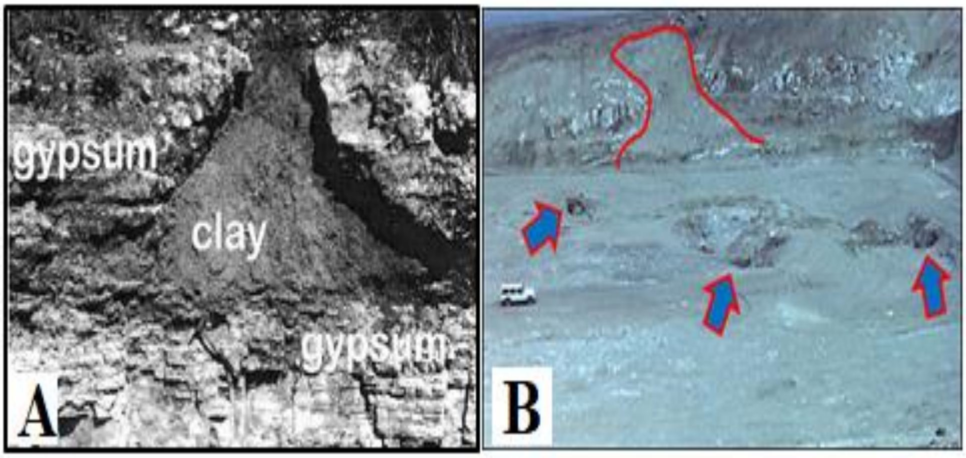


Figure 6: A) Gypsum- karst opening, now filled with clay deposited from through-flowing waters. Site is in the impoundment area of the abandoned Upper Mangum Dam. Image area is 2m wide [16]. B) A site 30km east of Mosul Dam. Clay filled karst in gypsum.

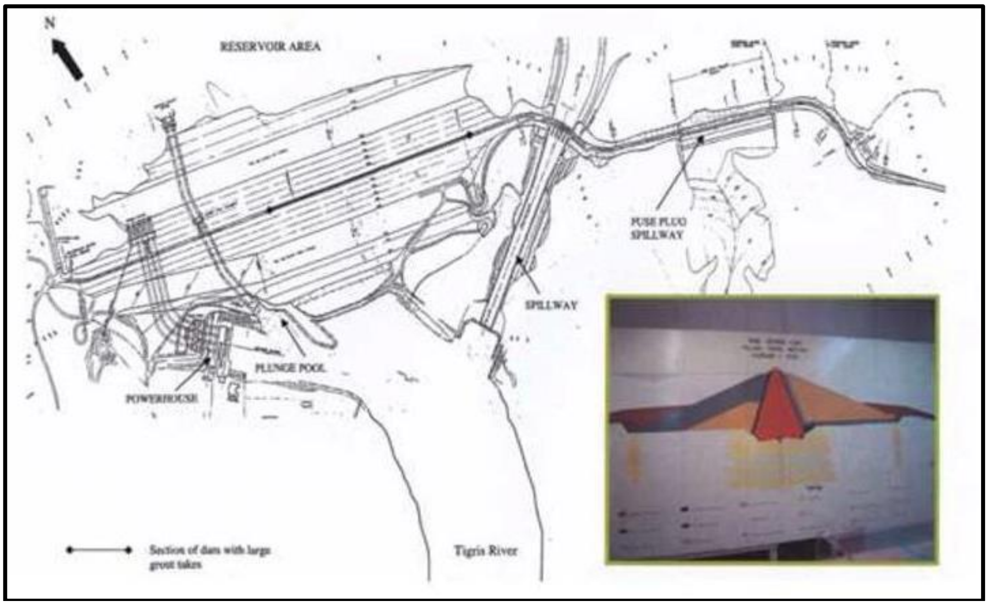


Figure 7: Mosul Main Dam – Layout and cross section.

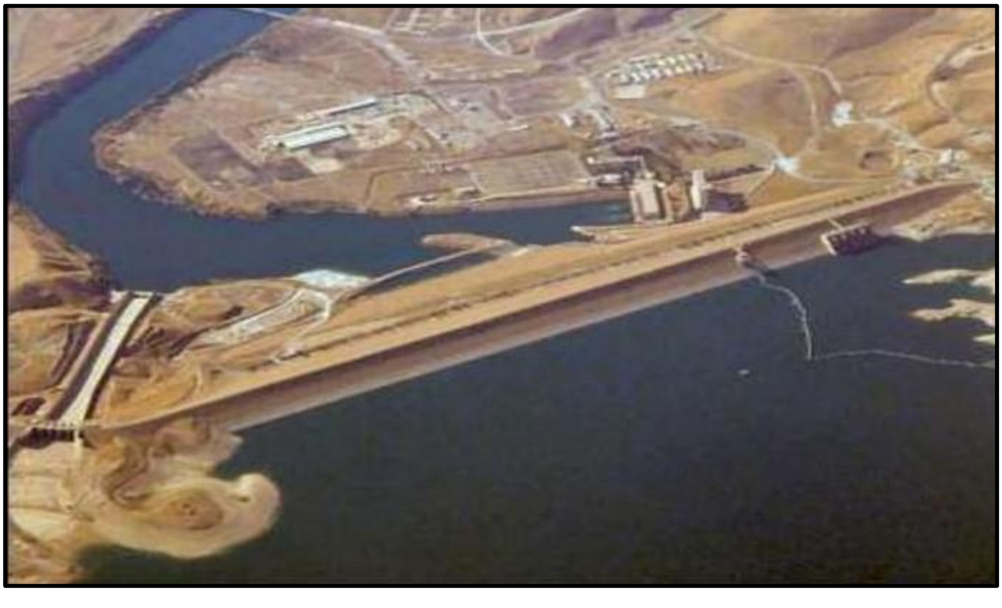


Figure 8: Aerial view of the Main Mosul Dam and appurtenant structures.

The source of problems is related to geological factors which are,

1. The karsts prevailing in the dam site and in the reservoir area.
2. The existence of gypsum/anhydrite rock formations in dam foundation alternating with soft marl layers and weathered and cavernous limestone beddings.
3. The presence of an extensive ground water aquifer which affects considerably the ground water regime in the central part and right bank.

Figure 9, shows the extent of the karst's phenomena in the form of sinkholes upstream area of the dam and in the reservoir.

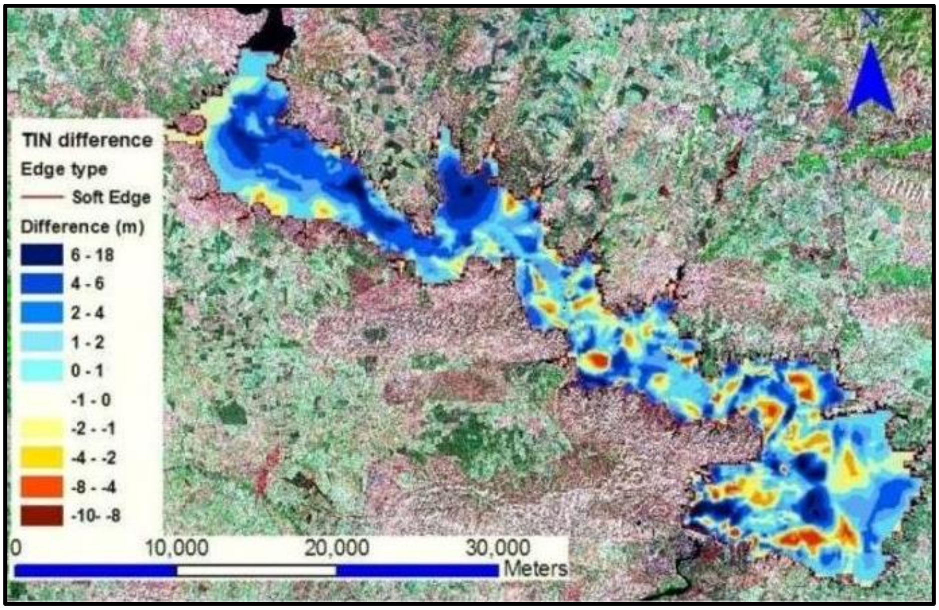


Figure 9: Sinkholes present in the Reservoir.

The dam foundations itself is very complicated and the combination of gypsum/anhydrite layers with the cavernous limestone gave rise to very favorable conditions for seepage and dissolution environment by the formation of highly developed conduits and caverns for the flow of ground water. The result was extensive dissolution of gypsum and gypsum anhydrite rocks present above and below these limestone layers. These dynamics caused the collapsing of whole layers of clayey marls into the underneath cavities forming beds composed of brecciated gypsum particles and anhydride blocks embedded into a loose clayey matrix as described before and shown in Figure 6. Four such layers were discovered during the geological investigations and were tagged as the (Gypsum-Brecciated) layers (GB0, GB1, GB2, GB3). These layers had thickness, which ranges between 8m and 16m; and were found at different depths, the last one was at depth of 80m below the river bed while the first day lighted in the excavation of the foundation of spillway chute ski jump on the left abutment of the dam. The GB layers proved to be very important due to their erratic behavior during the grouting of the deep grout curtain under the dam, where the deep this curtain was the main ante- seepage measure adopted in the design of the dam not only to reduce uplift to the safe limits, but, also to reduce the seepage flow under the dam to get saturated condition there and stop the dissolution of gypsum. This assumption was not realized as the GB layers did not accept all sorts of grouting solutions, or when it did these layers could not hold on the grout in the grouted zones and soon deteriorated and grout was washed out with more dissolution. This condition was discovered after river closure and diversion of the flow through the diversion tunnels, which made stopping the job practically impossible. The only way was to establish a continuous repair and maintenance grouting program, which continued from 1986 till mid 2014 with no hope of having any end to it. The quantity used during these years exceeded 95,000 tons of solid grouting materials. Although grouting is considered now as only a temporary remedial solution, while still looking for a permanent one, it is still very important to keep grouting works to increase the life of the project as long as possible and prevent a sudden failure. The grouting works, however, were forced to halt due to the occupation of ISIS to the site on 8th August 2014. The works did not start immediately after recovering the site ten days later, but as the site was secured later by an American force, a United States Interagency Team led by the USACE engineer`s and specialists started through investigations of the foundation conditions including, settlements, lateral displacements and checking of seepage water quantities and quality to find there was grave deterioration of the grout curtain, increased dissolution of gypsum and loss of material from the foundations.

This work took the whole of 2015, the final report of the findings was issued at the end of that year. By using the Screening Portfolio Risk Analysis (SPRA) procedure, which was developed and used by USACE since 2005 for Dam Safety rating of their portfolio of dams, they concluded that Mosul Dam was the most unsafe dam compared to all of the USACE dams. In fact, it showed that Mosul Dam was in a state of extreme and unprecedented high relative risk when compared to all the dams, which belonged to USACE, and it was on the verge of failure. These findings, and due to the pressures exerted by the US Government on the Iraq Government, a new grouting contract worth $296 million to conduct grouting and maintenance on the dam for a period of 18 months was signed in March 2016 with a foreign firm to restore foundation condition as before 2014. This work was completed on schedule after an extension of time to train the Mosul Dam personnel to continue the work for an indefinite time in the future. It may be concluded that, the karst hazard in this site was not figured out completely, and if the interpretation of the geological conditions during the many investigations campaign was correct, and if the behavior of gypsum and gypsum breccia was properly understood, then the designers should have never relied on the grouting curtain an as ante- seepage measure, but should have used a diaphragm wall. Even a little quantity of seepage through 80% efficient grout curtain was sufficient to cause continued dissolution as saturation condition could not be established.

In spite of the repeated repairing and maintenance grouting of the deep grout curtain, it cannot put an end to the karst dissolution at the site around the dam and in the reservoirs. Many sinkholes have formed so far just downstream of the dam which forced the authorities to limit the maximum operational water level to elv. 319m. a.s.l. instead of the maximum designed water level of 330m a.s.l. and so loosing sizable volume of the live storage and the benefits that are obtained from it with it. During the period 2014-2019, Mosul Dam problems and its safety risks became worlds' news on the media, and several hundred news reports and technical papers were published. Many such papers were written by the author and a technical team from Luleå Technical University of Sweden which give complete technical reviews and analyses of these problems; it is highly recommended to refer three such papers to the reader's attention for full understanding of this so far unique case of karst hazards in dam's construction. These papers appear in the references list, [17], [18], [19], [22]. Karst hazards in limestone are not less problematic than those in gypsum karst, as far as dam's safety and usefulness are concerned. One good example is the case of the Boljunčica reservoir in the Istria peninsula (Croatia).

This reservoir is impounded by the construction of the 35m high Letaj concrete arch dam on Boljunčica River. The dam was completed in 1970, but the filling of the reservoir with water started only in 1973. The reservoir is situated at the contact zone between water impermeable Eocene flysch and deep Eocene and Cretaceous limestone. The bottom of the reservoir is partly covered by both flysch and Quaternary deposits. The water volume of the reservoir at the spillway elevation of 93.00m a.s.l. is about 6.5 x 10^6m^3. This is a multipurpose reservoir, which was built in order to protect the fertile downstream Cepić polje from flood, to store water for irrigation and to control sediment transport. In order to prevent water leakage, a grout curtain was built in the limestone below and around the Letaj dam. The thickness of this grout curtain is about 20m. The main purpose of grout curtains is to change the hydrogeological characteristics (reducing the permeability) of the rock mass. Constructing a grout curtain in a karstic environment with high random distribution of karst features contains some uncertainties and surprises that cannot be excluded. The grout curtain developed around Letaj dam did not protect the reservoir from water losses. Up to 50 covers collapsed sinkholes have been found in the area. They are oriented along the eastern edge of the reservoir at the contact zone of flysch and limestone; their locations are shown in Figure 10.

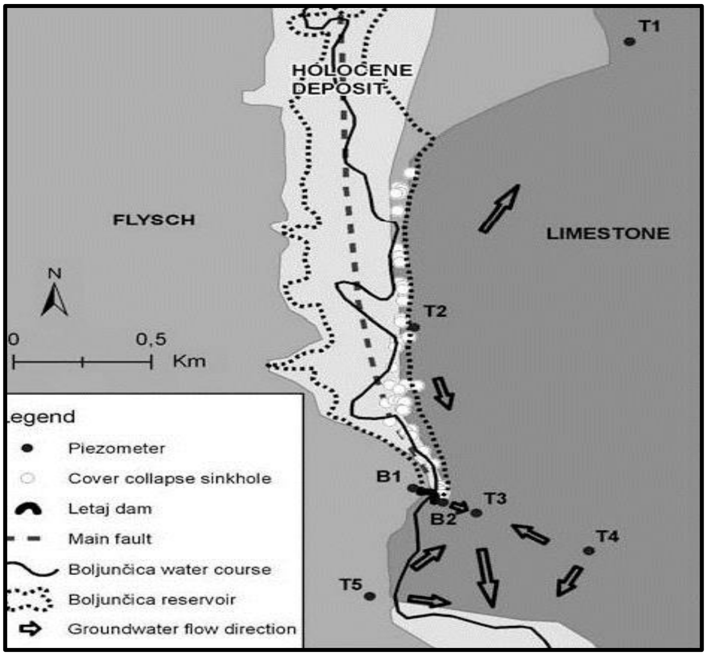


Figure 10: Map of reservoir with designated geological structure piezometers, cover collapse sinkholes and groundwater flow directions.

The minimum sinkhole`s elevation is 75m a.s.l., while the maximum reaches 89m a.s.l. The photographs of Figure 11a and 11b show two different shapes of covered collapse sinkholes. It is obvious that they serve as swallow holes, i.e. water quickly flows through them to the karst underground. It should be stressed that after each filling and emptying of the reservoir new covered collapse sinkholes appeared. Two different shapes of covered collapse sinkholes at the bottom of the Boljunčica reservoir are shown in Figure 11.

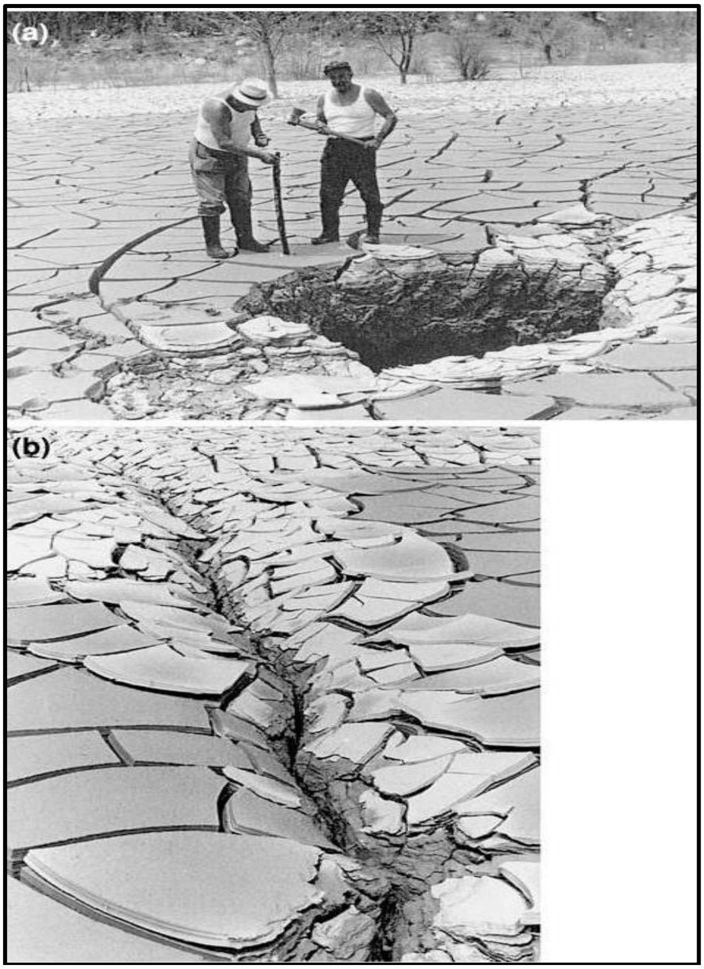


Figure 11: Photograph of two different shapes of covered collapse sinkholes at the bottom of the Boljunčica reservoir.

This case represents an unsuccessful case of building a reservoir in karst, caused mainly by insufficient geological, hydrogeological and hydrological investigations. Owing to the extreme heterogeneity of the karst underground forms, it is very hard to obtain reliable local information, parameters and conclusions on water circulation processes. It should be stressed that in different karst areas water losses from the reservoirs will become manifest in different ways. Karstification below dam sites and at the reservoir bottoms may increase water losses in the future. This is probably a case of the Boljuncˇica reservoir. The basic problem to recognize and explain the information contained in groundwater data. The best and probably only solution is holistic interdisciplinary cooperation amongst numerous experts [20]. Sinkholes, in addition to causing loss of valuable storage water, can pose an extreme threat to the stability of dams, if the happen to develop very close or in the foundation of such dams. In the case of Mosul Dam this hazard was considered thoroughly. In Potential Failure Mode Analysis (PMPA), the development of sinkholes close (refer back to

Figure 9) or under the dam figured prominently in three Potential Failure modes which are abstracted from a paper entitled “Risk Management Concepts in Dam Safety Evaluation: Mosul Dam as a Case Study” written by the author and a colleague from the Luleå Technical University of Sweden [21]. The following details, taken from Table 6 on page 648 of this paper, are summarized from these PFMs;

Potential Failure Mode N1

Cavity/cavern forms u/s at depth 0-10m below foundation of Mosul Dam in GB1, GB2, GB3 and possible collapse of cavern and connecting directly to reservoir, flow velocity increase leading to increased dissolution and internal erosion. Embankment is compromised by core collapse, loss of upstream shell dam breaches. Adverse conditions are: GB1, GB2 had been repeatedly grouted, sinkholes upstream had developed; blanket grouting does not extend under shells and doubts on the effectiveness of consolidation grouting; dam could fail if erosion starts at the dam/foundation interface. Effect of grout curtain is temporary and needs repeated grouting.

Potential Failure Mode N2

A large 5 to 30m diameter cavern forms u/s at depth 10-60m below foundation of Mosul Dam in GB1 progressing to foundation/dam body interface connecting to the reservoir. Increased inflow into foundation and increased dissolution and wash out of materials, embankment compromised, and collapse of section into cavity and dam breaches: adverse conditions: high grout takes were recorded in GB1, sinkholes upstream had developed; blanket grouting does not extend to this depth and no blanket under u/s and d/s shells, effect of the grout curtain temporary and needs repeated grouting.

Potential Failure Mode N3

A very large 30 to 40m diameter cavern forms u/s at depth of 60-80m below foundation of Mosul Dam in GB0, and possible collapse progress to the foundation/dam body interface connecting to the reservoir, leading to increased dissolution and internal erosion embankment compromised by loss of critical components and collapse of a dam section and dam breaches. Adverse conditions: GBO unit has 15-20m thickness with repeated high takes grouting; sinkholes have developed u/s and d/s of the dam, with only grouting works in the uncertain deep curtain, there may not be visible warning signs of deterioration until too late, dam section may collapse leading to dam breach.

Construction of dams in the karst region can be very challenging task and requires thorough geological and hydrogeological investigations and the recruitment of expert in these fields.

In the case of Mosul Dam, large volume of geological information was accumulated during the investigations, but it is unfortunate to see that some of the basic facts were not interpreted correctly [22]. This applies to the incorrect correlation of the encountered beds in the exploration boreholes and miss-understanding of the actual stratigraphic succession at the dam site. This misinterpretation contributed to misleading results regarding the true karst zones, depths and the type of rocks and their thicknesses in the foundation zone and surrounding area. As a result, the dam was placed on problematic foundations consisting of brecciated and highly karstified gypsum/anhydrite rocks and/or conglomerates in which gypsum forms the main constituent as cementing materials. Karstified beds were not recognized in some depths and were described as normal marl and/or breccias. Moreover, Terra Rosa; a good indicator for karst forms was interpreted as “bauxite” [22]. This also added to the use of improper method of foundation treatment by adopting a deep grout curtain as the main anti-seepage measure instead of using a more positive measure by constructing a diaphragm wall [22].

In response to the great number of karst failures and hazards in the previous years, the engineering problems in karst have been the main topic of several conferences and meetings in the last decades that resulted in interesting volumes and special issues. From 1984 to 2011, twelve Multidisciplinary Conferences on Sinkholes and the Engineering and Environmental Impacts of Karst have been organized in the United States and in Europe. Since 2004, problems related to karst environments have been the subject of specific sessions at the General Assembly of the European Geosciences Union within the framework of the Natural Hazards Program. In many cases, the most significant outcomes from these sessions have been published as special issues of peer-reviewed journals or as a book. Notwithstanding these efforts, little attention is given to these issues in karst landscapes, and few regions of the world fully understand or appropriately manage karst areas [23].

4. Hazardous conditions in dam’s foundations

Foundation is a critical structural element in the safety of any dam, which serves two purposes:

1. To provide structural stability and sufficient stiffness to limit deformations to within acceptable behavior patterns under the weight of the dam and the forces acting on it, and to maintain this integrity under the conditions that exist and/or can be expected to develop over time.
2. To control seepage under the dam with respect to flow quantity and quality, uplift pressures, and erosive stresses. If one of these functions is not sufficiently addressed, the dam’s performance may be impaired to the point that the dam is unsafe.

The foundation conditions at any particular site are also a determining factor in the selection of the type of the dam, and its safe operation. In the context of dam safety, the design and supporting geological investigation are important critical controls’ elements that reduce risks. Accordingly, the dam itself is planned, and quality control can be exercised through design and construction specifications, but the foundations’ conditions are determined by nature and are subject to the inherent heterogeneities and potentially complex conditions, which should be discovered by proper geological investigations. In any meaningful geological investigation of the certain dam site, many questions are to be answered to avoid unpleasant surprises with respect to geologic hazards, during construction or operation. Typical questions are, presence of hazardous geological structure under the dam, faults, their extent and nature, degree of weathering in rock formations under the dam, presences of weak layers, which represent weak shearing plains…etc.

Faults in Dams Foundations

One striking case of undiscovered faulting foundation's leading to dam failure is Malpasset dam catastrophic failure in France on 2nd December 1959. The dam was a 66-m high thin arch dam situated on the Reyran River 10km upstream of Frejus in the Cannes District, France. The arch was 1.5m thick at the crest and 6.7m thick at the base, refer Figures 12a and 12 b. The right abutment consisted of massive rock, but the left abutment depended on a thrust block protected by a wing wall due to the topography of the site. An overflow spillway, 30 m long and capable of discharging 178m^3/s was located in the central portion of the crest [24], [25], [26]. The reservoir had a capacity of 51.4x10^{6}m^3 at the maximum water surface.

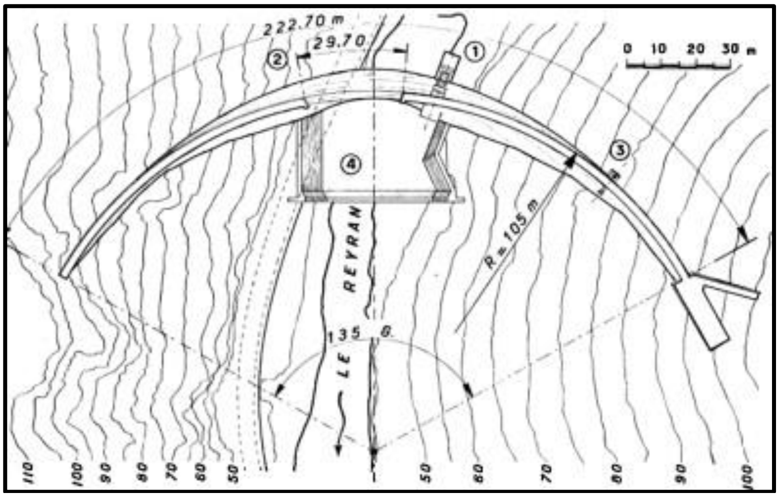


Figure 12a: Plan of Malpasset Dam [24].

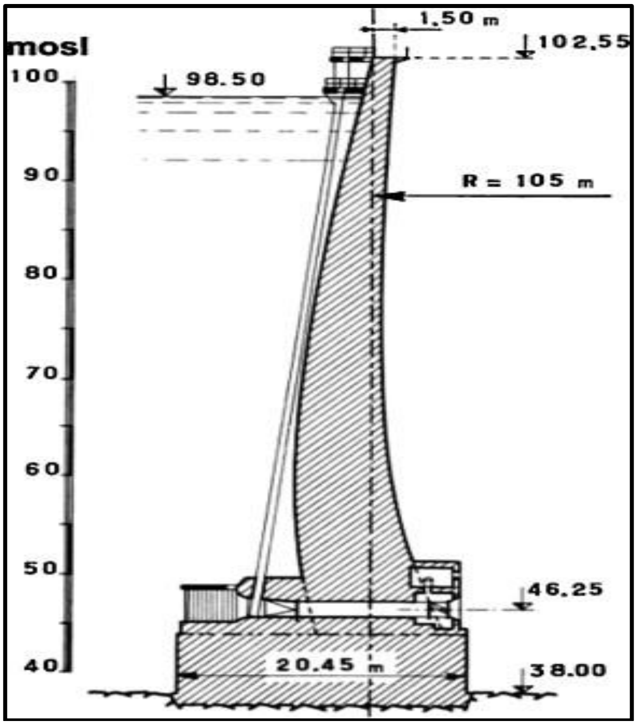


Figure 12b: Cross Section of Malpasset Dam [24].

The foundation was described as a “synclinal carboniferous zone enclosed by the metamorphic horizons of the base Massif of the Esterel”. Exposed at the surface and in the investigation tunnel, were numerous stratified mica schist layers, which could be crumbled with bare hands. A fault zone was present in the left abutment, unidentified until after the failure, refer Figure 13. The foundation contact was grouted with blanket holes to a depth of about 5m. A grout curtain was not considered necessary due to the low permeability of the rock below the 5m depth. No drainage had been provided in the dam or foundation, and no instrumentation, other than surface measurement point was installed.

After the failure, the flow of water has cleaned the slopes perfectly from any loose or even weathered material, the rock mass structure appeared very heterogeneous and crisscrossed by joints at all scales and in any direction. Only the failure daylighted two features of the rock mass which proved instrumental. A huge block of foundation rock was missing at the left half of the dam arch, leaving an excavation in a dihedral form limited by two sub plane faces, as visible in Figure 14. The downstream face is a true fault plane with a thin cover of crushed rock (fresh scratches on the surface proved the whole block had moved upwards). Its upstream face looks like a set of tears along two or more foliation surfaces, without any crushed rock. Neither the fault nor the foliation had been recognized before, but it strikes perpendicular to the valley axis and its dip is about 45° upstream would have considered it as perfectly neutral with respect to the thrusts received from the dam; the continuity of rock foliation had not appeared either, within the so heterogeneous structure of the gneiss. So, a geometrical trap was in place, waiting for a force susceptible to move the block, which will appear later. The cleaning action of the flow made the fault path visible on the right bank and daylighted its cross section at either bank toe, refer Figure 15.

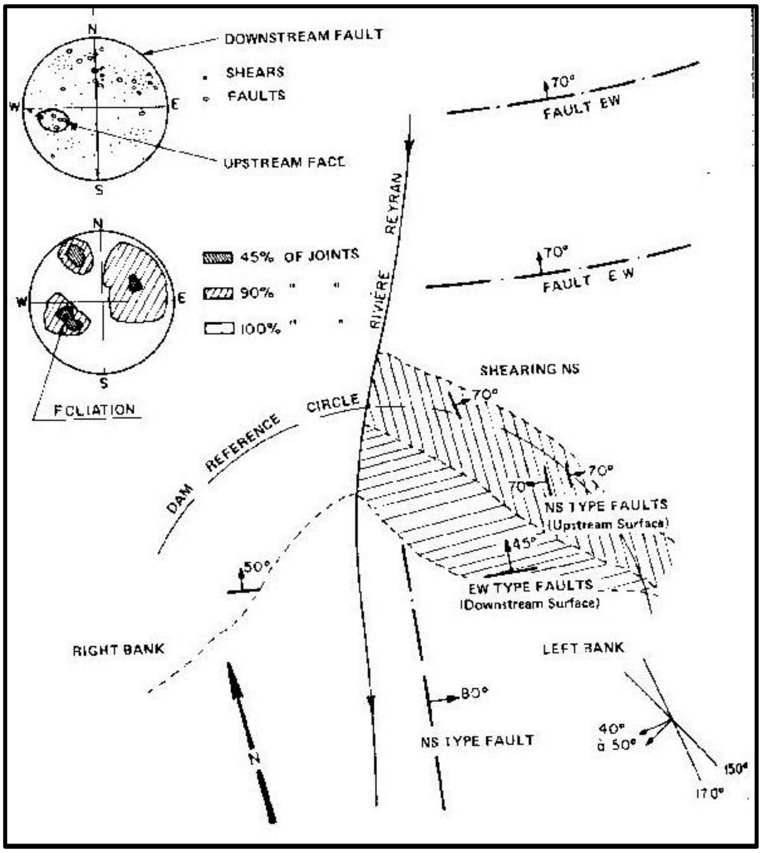


Figure 13: Geology of the Malpasset Dam Site (after Londe, 1987) [25].

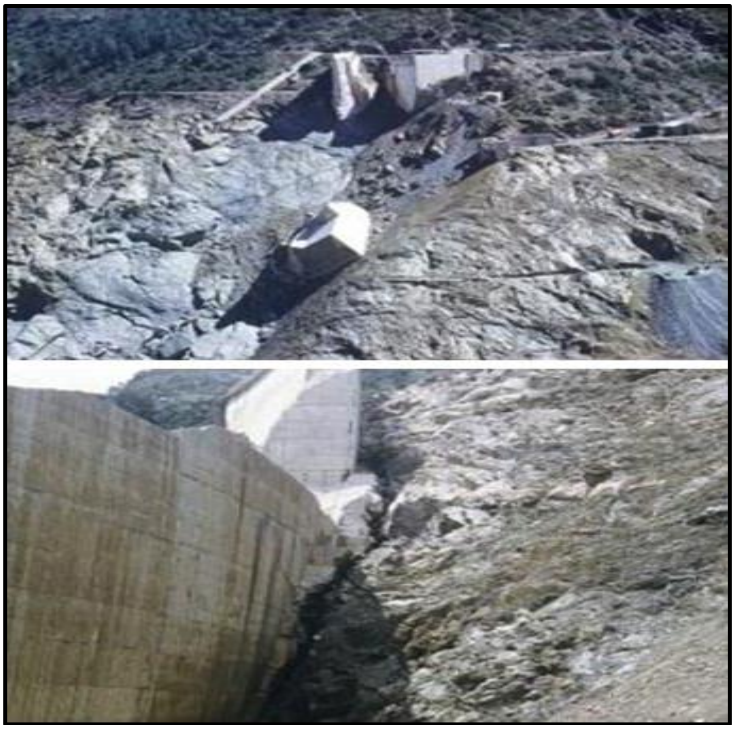


Figure 14-Top: the “dihedral” excavation with half of the thrust block fallen after the flow. Bottom: a wide crevice is open just upstream of the concrete arch, wider at the base and closing more higher [24].

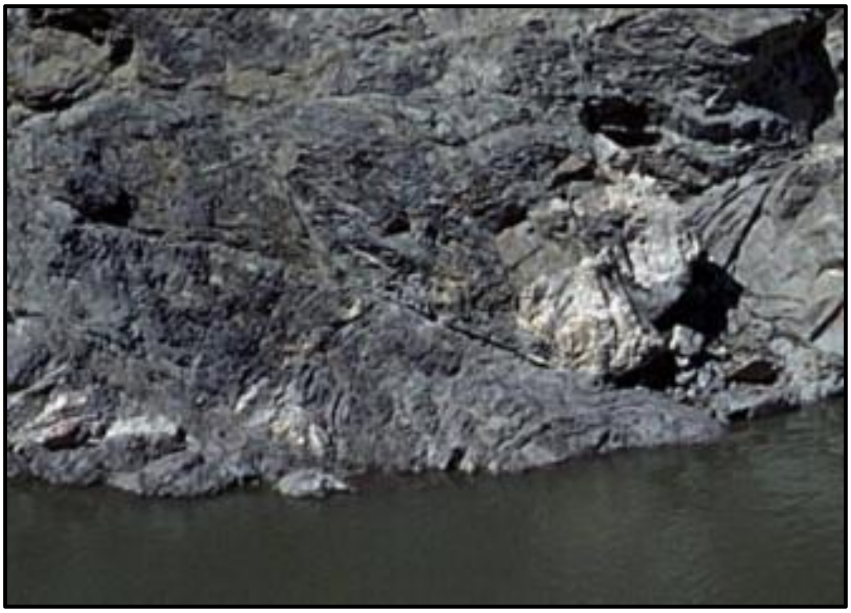


Figure 15: Close view of a cross section of the main fault on right bank. The finely crushed borders of the fault zone are well visible.

Post-failure review uncovered no problems of the arch structure. The design and construction of the dam were well-executed and in accordance with current standards. Material tests and trial load analysis indicated the dam structure could easily withstand loads associated with the reservoir filling. Further investigation, however, determined that the failure resulted from particular attributes of the foundation material and the geologic structure (discontinuities) that were not recognized. All measured movements and post- failure evidence pointed; however, to sliding on the fault as the cause of failure.

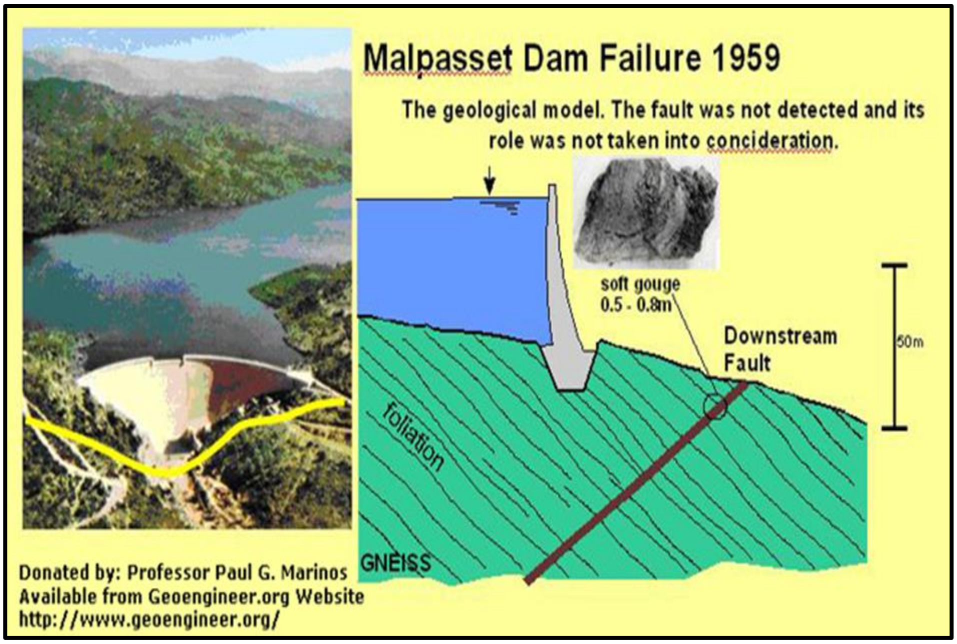
After thorough investigation of all factors, it is believed that the failure was indeed caused by sliding on the upstream dipping fault plane. Large uplift forces developed on the upstream foliation shear; for one of the several reasons or a combination of reasons:

1. The stresses generated in the rock extended to great depths, decreasing the rock permeability and forming an underground dam.
2. Tensile and shear stresses at the upstream face opened the foliation shear planes and allowed water pressures to easily penetrate to great depths, or
3. Clayey materials along the foliation shear plane created a natural dam in the foundation.

As the reservoir water rose, the loading increased until the block began to move. The arch tried to redistribute the loads higher to the thrust block, but under the extreme overload, the thrust block moved about 0.8m downstream separating from the attached wing (foot) wall. Without any place to transfer the load, the entire left side of the dam lifted and rotated downstream (all the while the hydraulic forces are increasing) until the dam ruptured. Professor Carl Terzaghi commented upon the Malpasset failure in February 1962 as follows:

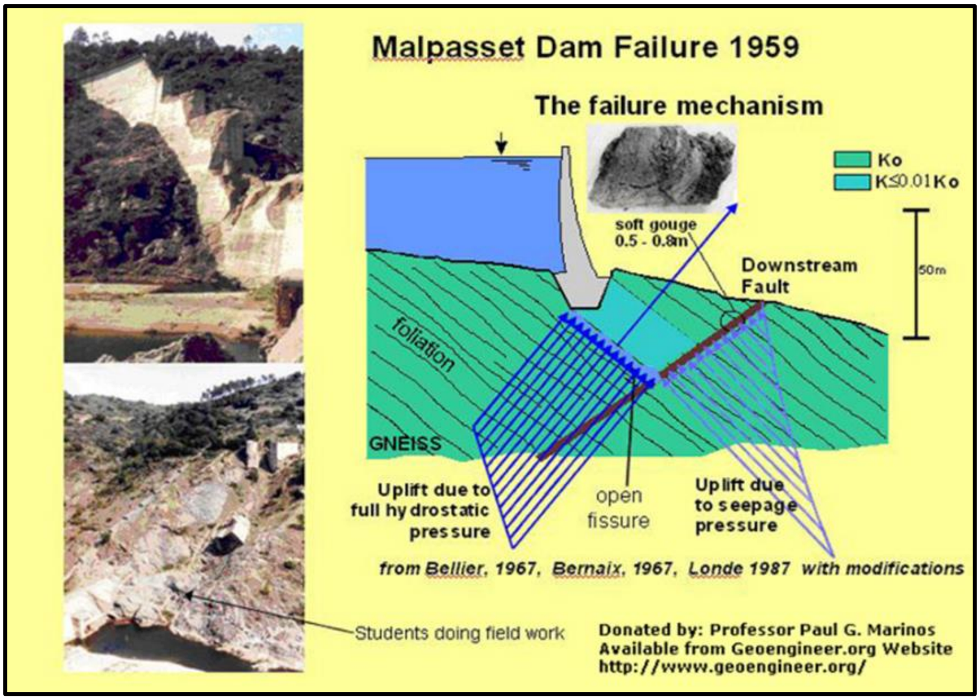
"The left abutment of this dam appears to have failed by sliding along a continuous seam of weak material covering a large area. A conventional site exploration, including careful examination of the rock outcrops and the recovery of cores from 2-inch boreholes by a competent driller, would show - and very likely has shown - that the rock contained numerous joints, some of which are open or filled with clay.", he adds;

"From these data, an experienced and conservative engineer-geologist could have drawn the conclusion that the site is a potentially dangerous one, but he could not have made any positive statement concerning the location of the surface of least resistance in the rock and the magnitude of the resistance against sliding along such a surface..." The development of the failure process may be better understood with the help of Prof Paul G. Marinos visualization shown in Figure 16 and Figure 17 [27].


Malpasset Dam Failure 1959
The geological model. The fault was not detected and its role was not taken into concideration.
soft gouge
0.5 - 0.8m
Downstream
Fault
50m
foliation
GNEISS

Donated by: Professor Paul G. Marinos
Available from Geoengineer.org Website
http://www.geoengineer.org/

Figure 16: Malpasset Dam Geological Model [26].


Malpasset Dam Failure 1959
The failure mechanism
🟩 Ko
🟦 K≤0.01Ko
soft gouge
0.5 - 0.8m
Downstream
Fault
50m
foliation
GNEISS
Uplift due to full hydrostatic pressure
open fissure
Uplift due to seepage pressure
Students doing field work

from Bellier, 1967, Bernaix, 1967, Londe 1987 with modifications

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Figure 17: Malpasset Dam Failure Mechanism [27].

5. Dams on hazardous weathered foundations

It is normal for dams' designers to look for the best possible geological conditions in any proposed site subject to other factors such as topography, construction materials availability and economic considerations. The selection of type of a proposed dam, however, is controlled mainly by the foundation condition(s) at the selected site as a primary factor. Generally, gravity dams require sound rock foundation, which may be reached after excavating and removing of fewer competent and unconsolidated layers to reach solid rocks, while arch dams need, in addition, good and massive rock abutments which often do not improve with distance excavated into the sides of the valley. Except for low dams of small gross weight, concrete dams are not built on unconsolidated deposits because of their low bearing strength. Larger dams constructed in whole or in part on unconsolidated deposits should without exception, be earth or rockfill dams with the capacity to adjust to settlement in the foundation materials. Dam safety in all cases must be insured, and if it is economically feasible, all hazardous material under the base of a proposed dam, which could cause excessive settlement and leakage, should be removed. If this cannot be done, the dam design should be modified to consider such material. Sometimes it may be necessary to remove material to considerable depths, or scale of loose masses of rocks on the abutments and in such cases, it rests with the engineering geologist to determine the expected depth of weathered or unsound rock or overburden that must be removed in advance of construction. If it is not feasible to remove all of the weathered unconsolidated materials, and in order to eliminate their hazards, the design may include some or all of the following measures;

a) Excavating and removing all low-quality materials for the purpose of preparing of a clean surface that will provide optimum contact with the dam materials, whether earth fill or concrete, to be placed on that surface. Therefore, excavations in bedrock should extend into firm, fresh rock. Any closely fractured zones extending downward, especially if containing soft altered materials such as clay gouge or other products of weathering, should be removed if feasible. For earth fill dams' cutoff trench may be excavated down into bed rock and filled with the same core material of the dam without the need to remove the material from the remaining part of the dam base, where scarifying the surface may suffice. Prolonged exposure of both earth and rock foundations to the atmosphere or to water frequently results in deterioration by hydration, dehydration, frost action, shrinkage, and expansion with changes in temperature. It is a good practice to protect reactive surfaces that will be exposed for long periods of time with bituminous materials or plastic sheeting, alternatively; original cover is not removed until final cleanup and just prior to placement of the dam.

b) Poor foundation conditions in bed rock formations are associated with close fracturing, weathering or hydrothermal alteration, or poorly indurated sedimentary rocks; therefore, treatment by dental concrete and blanket grouting may have to be done to some depth as decided by the geologist in order to consolidate these formation and reduce settlement to the minimum acceptable limits and at the same time hinder the penetration of seepage water at the contact surface.

c) c) Permeable foundations may pose a great risk to dams; seepage water under the dam must be reduced to safe limits, not only to reduce the seepage quantity, but also to reduce uplift. In addition to excavating a cutoff trench as in point (a), the construction of the deep grout curtain or positive cutoff (diaphragm) are other measures that shall be considered when required. Seepage water within soluble foundation, unless reduced to obtain saturation conditions, brings with it the hazards of dissolution of soluble materials and excessive settlement.

6. Foundations bedding with weak rock layers

Presence of thin layers of material of doubtful quality in the foundation of a proposed dam may require drastic changes to the design, if they are discovered at an early stage, or considering some stabilizing measures later on, otherwise the selected site may have to be abandoned to a better one, especially if such layers appear to be hazardous when wetted with water of the future reservoir in order to avoid sliding of the dam or excessive settlement. Overlooking such layers may lead to dangerous consequences and even failure. To clarify the point, the following case histories are presented. The first is the case of Mosul Dam (Iraq), and the other two are the two cases of St. Francis Dam (USA), and Austin Dam, Texas (USA).

Mosul Dam Case:

In this case, thin seams of plastic clay within the Lower Marl Series were discovered at an early stage of construction, which led to changing the earth fill embankment design by adding sizable toe weight at each of the upstream and downstream sides in order to ensure stability of the dam at an acceptable safety factor. During the ongoing geological investigations just before the start of construction, intersections of thin clay seem showing slickensides were recovered from drill holes. The excavation of two exploratory shafts on the right abutment confirmed the presences of such clay seams, while the large excavation cuts into Lower Marl Series on the right bank at the location of the power house, power intakes and guard gate chamber as well as the grouting gallery trench along the dam axis confirmed the presences of numerous clay seams at various horizons in the foundation whose thickness varied between few millimeters to one or two decimeters and showing signs of slickensides. Five prominent seams were indicated in the upper levels, but all of them were extending parallel to the beddings' planes, which indicated their sedimentary origin, and they continued at least over several hundreds of meters. Extensive laboratory testing was performed to stablish the strength parameters (φ, C) of these seams, and several stability studies (static and dynamic) with various sets of these values were conducted using sliding wedge analysis instead of the conventional slip circle analysis, passing each time through one of these seams. The obtained results showed generally low factors of safety against sliding in seams no. 1, 2, 3, and 4 (numeration from top to bottom), while failure was eminent in seam no. 5 under seismic loading. The decision taken by the designers, after long deliberations with the reviewing International Board of Experts, was to take the excavation of the dam foundation at the location of powerhouse, power intake structure, and guard gate chamber down to remove the first top four seams, and to add sizable toe weights at both upstream and downstream for the whole dam length in the form of berms in order to establish acceptable factors of safety at all sections of the dam. More than 500,000m^3 of extra excavation and more than 6.37 x 10^{6}m^3 of random fill in these two berms were added to the scope of work of the contract [28].

St. Francis Dam Case

This dam, an arch gravity concrete dam in Southern California, was completed in May 1926 but failed on March 12, 1928. It was 62.4m high situated on San Francisquito Creek, a tributary of the Santa Clara River. The dam was 4.9m thick at the crest, 53.3m thick at the base, and included a low wall extending 152.4m from the main arch along a narrow ridge on the right abutment. The length of the main portion of the dam was 213.4m along the curved crest. Eleven spillway openings, each 6.1m wide were located in the central portion of the crest. There were five 0.76m diameter outlet pipes at vertical intervals of 11.0m which were controlled by slide gates on the upstream face of the dam. The dam had no contraction joints or inspection gallery. The foundation was not pressure grouted and only drained under the center section which was the only section to survive. The reservoir had a capacity of 46.9 x 10^6 m^3 at maximum water surface and was to be used as a backup water supply for Los Angeles in case the flow of Owens Valley water was interrupted, Figure 18.

The foundation of the dam was comprised of two kinds of rocks. The canyon floor and the left abutment include mica schist known locally as “greywacke” shale. The schistosity planes were essentially parallel to the canyon wall, dipping toward the canyon at about 35º. The schist on this abutment splits easily and weathers readily. In many places, it has been sheared, roughly parallel with the planes of laminations. In these shear zones, the rock has been reduced to flaky materials that are quite fragile. The steep slope of the left abutment (East Abutment) can be attributed to the laminated structure. The rock can resist large loads perpendicular to its planes of lamination, but it is weak against loading parallel with those planes. In the shear zones, the rock has been reduced to flaky material that is quite fragile. The upper half of the right side (west Abutment) of the foundation was a red conglomerate. The contact between the two formations was a fault which, at the dam site, had a strike approximately parallel with the stream and dipped into the right abutment at about 35º and outcrops about 15 meters above the channel. The conglomerate was laced with fractures, some of which contain gouge or gypsum. Many of the pebbles in the conglomerate have been sheared. The whole mass has been so crushed that it has lost much of its strength. In the dry condition, fragments of considerable size can be broken out and trimmed with a hammer. When immersed in water, however, absorption proceeds rapidly, air bubbles appear, crumbling begins, the water becomes turbid with suspended clay, and usually in less than an hour a specimen with the size of a baseball disintegrated almost completely.

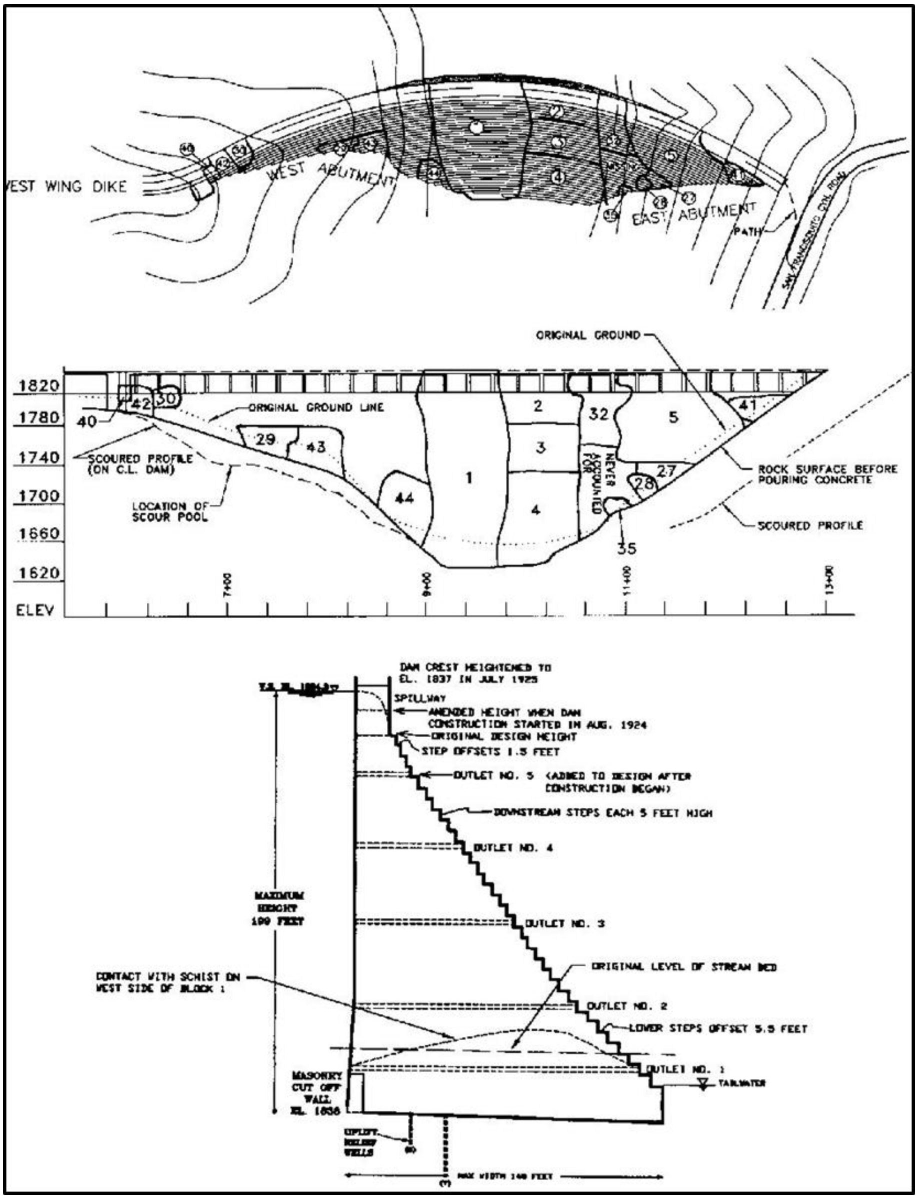

WEST WING DIKE
WEST ABUTMENT
EAST ABUTMENT
PATH
SAN FRANCISQUITO CYN. ROAD
ORIGINAL GROUND
1820
1780
1740
1700
1660
1620
ELEV
ORIGINAL GROUND LINE
SCOURED PROFILE (ON C.L. DAM)
LOCATION OF SCOUR POOL
NEVER ACCOUNTED FOR
ROCK SURFACE BEFORE POURING CONCRETE
SCOURED PROFILE
1
2
3
4
5
27
28
29
30
32
35
40
41
42
43
44
7+00
9+00
11+00
13+00
DAM CREST HEIGHTENED TO EL. 1837 IN JULY 1925
SPILLWAY
AMENDED HEIGHT WHEN DAM CONSTRUCTION STARTED IN AUG. 1924
ORIGINAL DESIGN HEIGHT
STEP OFFSETS 1.5 FEET
OUTLET NO. 5 (ADDED TO DESIGN AFTER CONSTRUCTION BEGAN)
DOWNSTREAM STEPS EACH 5 FEET HIGH
OUTLET NO. 4
MAXIMUM HEIGHT 190 FEET
OUTLET NO. 3
ORIGINAL LEVEL OF STREAM BED
CONTACT WITH SCHIST ON WEST SIDE OF BLOCK 1
OUTLET NO. 2
LOWER STEPS OFFSET 5.5 FEET
OUTLET NO. 1
TAILWATER
MASONRY CUT OFF WALL EL. 1636
UPLIFT RELIEF WELLS
MAX WIDTH 140 FEET

Figure 18: Plan, Profile and Section of St. Francis Dam.

Soon after the failure, the surface of conglomerates at the dam site showed marked signs of softening due to saturation. After a few days of drying, this same rock exhibited a smooth surface when broken and some of the samples rang under the hammer. Of two specimens selected from the firmest part of the eroded foundation, one fractured while being readied for testing and the other was found to have a compressive strength of only 3606 kilopascals (523 pounds per square inch). This was no more than one-fourth the strength of the concrete in the dam.

The dam was placed across the fault with full knowledge of its existence. It did not undergone any movement during the life of the dam. There evidently was no basis for blaming the disaster on a fault shift, even though the foundation at this point was exceptionally weak. Along the contact was a band of serpentine, which was so softened by water after the failure that it could be dug out by hand. The fault contains a clay gouge as thick as 1.2m. When dry, it is rather hard, but upon wetting it becomes soft and plastic.

With such a geological setting, the collapse of the St. Francis Dam was inevitable; unless water somehow could have been prevented from entering the foundation. This; however, was impossible. Pressure grouting drain wells, galleries, and deep cutoff walls used at other dams to control seepage probably could not have been fully effective at this site, although they might have postponed the day of failure.

In view of the many deficiencies of the site, the survival of the structure for two years is remarkable indeed. Storage of water began on March 1, 1926. From the beginning of 1928, when the water surface elevation was 555.0m a.s.l., storage was increased until March 5, when the reservoir was essentially filled to capacity, at elevation 559.2m or 0.076m (below the spillway crest). It was maintained at that level until the failure at a few seconds past 11:57 p.m. on March 12, 1928.

Photographic evidence and the reports of witnesses confirmed that there was not much seepage through the dam. Some cracking appeared in both the gravity mass and the wingwall, and minor seep were noted in certain of these places, but none of this flow through the structure itself was viewed as alarming. However, more significant seepage developed through the dam’s foundation. As the reservoir level rose, the magnitude of this discharge increased considerably. It had reportedly reached a maximum of nearly 57 liters per second in the afternoon before the disaster. Seepage along the contact between the dam and its foundation, especially on the west abutment, was noted as soon as impoundment began. Two or three weeks before the catastrophe, flow under the wingwall increased, and some drains were installed. In the 24 hours before the collapse, the seepage in that area accelerated. On the morning of March 12, 1928, inspection of seepage water showed that it was clear, which indicated that foundation material was not being eroded from beneath the dam. There was nothing in the performance of the structure that they judged to be hazardous.

The disaster was subjected to several inquiries. All attributed the cause of the failure to defective foundations. A committee selected by the District Attorney of Los Angeles County on about March 15, 1928, concluded the following:

a) The defective foundation material referred to as a conglomerate, became softened by absorption and percolation of water from the reservoir and was softened by hydrostatic pressure and pushed out from under the dam structure, permitting a current of water at high velocity to pass under this sector of the dam. This current, by eroding the soft foundation material, quickly extended the opening under this portion under the structure to such an extent that a part of the Westerly section of the dam collapsed through lack of support.
b) Following failure in the westerly sector, the escaping water swirled across the downstream toe of the dam to the easterly wall of the canyon below the easterly abutment, causing a slide which broke the already weakened bond between the easterly sector and the side wall of the canyon, allowing the easterly sector of the dam to collapse.
c) One portion of the easterly sector of the dam fell and now lies upstream from the original face of this sector, indicating that this portion failed after the water level had dropped enough to materially relieve the pressure from the upstream face.

Other consultants retained by the Santa Clara Water Conservation District filed an engineering and geological report on April 19, 1928, stating:

a) The old slide against which the dam rested at the east which is composed of mica schist bedded at an angle of, or approaching 45° with the horizontal and practically paralleling the general slope of the canyon wall, and shattered by previous movement due to lubrication with water from the reservoir; as well as with infiltering rainwater, offered only insecure support to the dam, and this was rendered more precarious by the adoption of a design which did not include adequate foundation drainage.
b) At the west end of the dam, the material on which it rested is more or less pervious and softens when wet. It remains doubtful whether any precautions (such as cut-off walls and weep-holes) could have made this a satisfactory foundation.
c) The appearance, as early as January 1928, of cracks in the dam other than temperature cracks, indicated a movement of the dam. Apparently, the significance of these cracks was overlooked.

We can learn many lessons from the St. Francis Dam failure, in our opinion these may be the following:

a) The geology of the site was not well understood, especially with respect to this type of hazardous rock materials and their behavior under water. Sound geologic input should have been obtained by employing an experienced geologist, and by performing full scale geological testing of representative rock samples to get all relevant properties.
b) Treatment of the foundation and abutments by grouting to an appreciable depth could have been useful to consolidate and strengthen the rock mass in both the abutments and floor of the gorge and reduce seepage and percolation through the beddings

c) The design engineers should have had good understanding of the concepts of uplift and effective stress which they (he) seem to be lacking [29], [30].

Austin Dam, Texas (USA)

The Austin Dam, also referred to as "The Great Granite Dam" failure was a catastrophic dam failure near Austin, Texas that killed several dozen people in 1900. The destruction of the dam drained the Lake McDonald reservoir and left the city of Austin without electrical power for a number of months. This masonry gravity dam was constructed on the Colorado River, Texas, about 4 km upstream of Austin. The dam was 19.8m high, 342.9m long, and had a total base width of 20.1m. It was constructed of limestone masonry and faced with granite blocks. The mortar of the masonry core had a compressive strength of 16.00MPa. A powerplant was constructed downstream of the left abutment, and the central section had an ogee-type shape to allow water to flow over the dam during flood events. The dam is founded on interbedded limestone that dips downstream 3 or 4m per 100 m. The beds are up to a few feet thick and alternate between hard and soft layers. The rock is extensively fractured and contains much open or clay filled seams. A shallow 1.2m deep cutoff trench was excavated at both the upstream and downstream foundation contacts into the rock. Flowing springs were encountered in both the riverbed and abutments during construction, Figure 19. Because the dam was faced with granite blocks, it was named at the time the "Great Granite Dam". Shortly after completion in 1894, problems were encountered at the powerplant head gate structure due to water flow along a fault beneath the structure. The structure was badly damaged (exact failure mode unknown), necessitating costly repairs. On April 7, 1900, a heavy rainstorm resulted in 3.4m of water going over the crest of the dam. At that time, about 134.1m of the left side of the dam slid downstream about 12.2 or 15.2m. This section of the dam maintained its line and vertical position. It then broke into two pieces, which remained intact for about 1 hour before breaking up and washing away, refer Figure 21. The water flowing over the structure had eroded the toe and exposed daylighting bedding planes. The strength of these planes was insufficient to resist the loading [29].

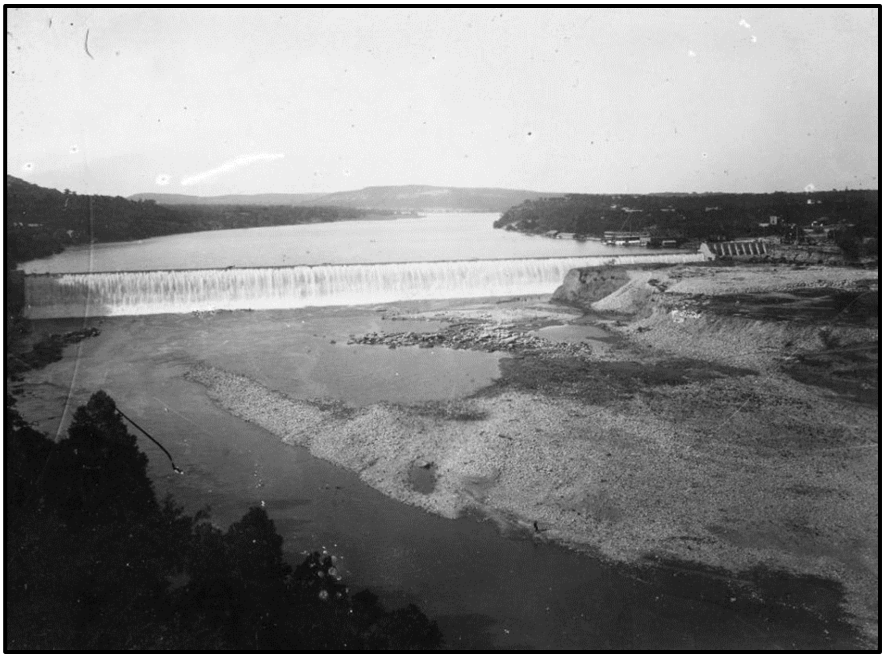


Figure 19: View of the completed Austin Dam, 1890's. (LCRA Archives, W00108) [31].

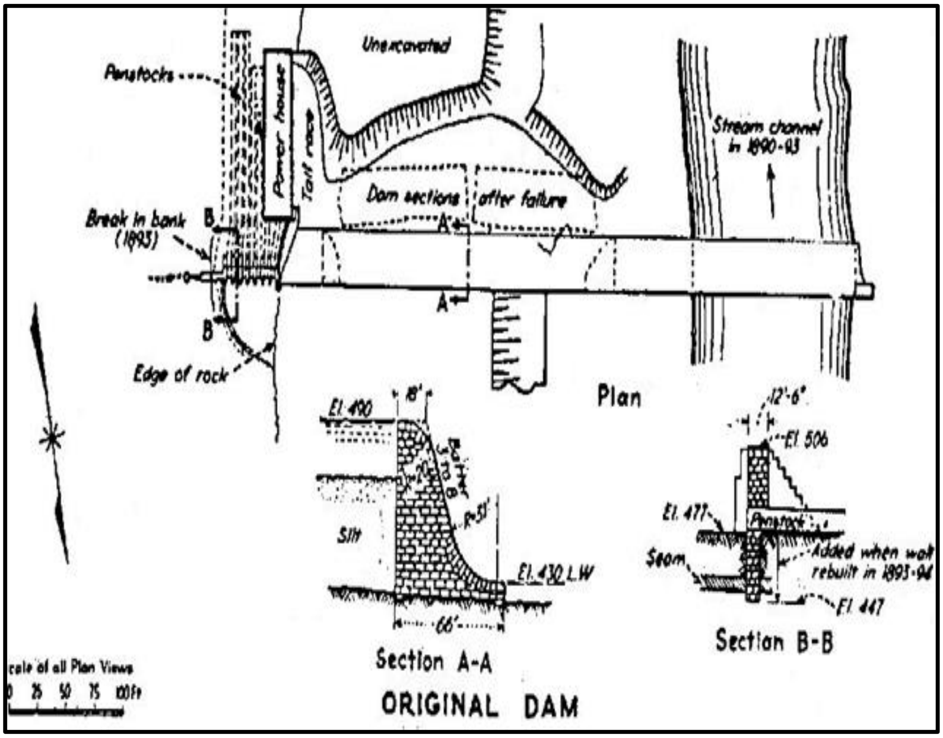


Figure 20: Austin Dam, Texas, plan and sections showing conditions after flood (1900) [29].

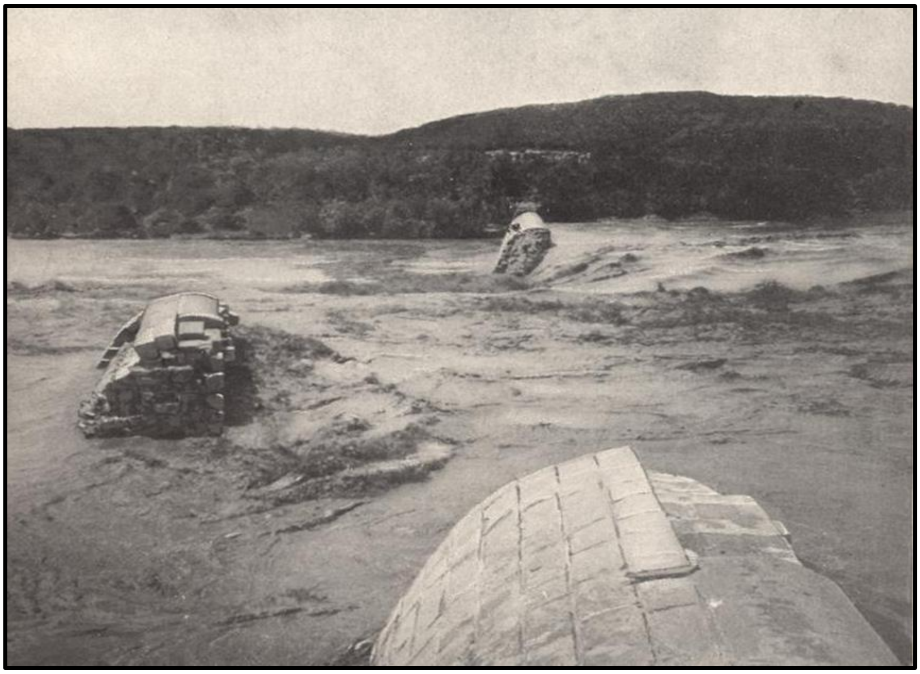


Figure 21: A view of the dam one hour after break [32].

Twelve years later, rebuilding of the dam began. A hollow reinforced concrete buttresses dam was built in the gap created by the 1900 failure. The buttress footings were carried to the first hard rock layer encountered, shallow cutoff walls were constructed, and foundation grouting was performed to depths from 1.5 to 3.4m. The crest of the rebuilt masonry dam was 2.7m below the old masonry dam. A 4.6m high spillway gates (apparently radial gates) were constructed on top of the buttress dam, and 1.8m high crest gates (some type of slide gate, flap gate, or stoplog) were placed on the masonry dam. Reconstruction was completed in 1915, but soon afterward, another flood destroyed 20 crest gates and blocked the tailrace and draft tubes of the powerhouse with debris. In 1917 the dam was investigated by Daniel W. Mead, a consulting engineer called in by the City of Austin. He recommended extensive repairs to the foundation and superstructure, but no action was taken. In 1935 a flood greater than any of record, and three times as large as that which caused the original failure, resulted in water flowing 7.6m over the crest of the dam. The flood tore out all but three of the crest gate piers on the old masonry dam, and more than half of those on the buttress dam. A portion of the buttress dam crest slab broke away, and over 200 feet (61.0m) of the downstream face slab tore away. Going back to understand what originally had happened indicated that the foundation of the original dam was virtually on the surface of the riverbed rock. The foundation here consists of nearly horizontal layers of Edwards limestone, which is extensively fractured and contains many open or clay-filled seams. Recalculation of the stability of the dam for the overtopping conditions which were occurring at failure of the original dam showed that tension existed at the heel of the dam when a linear uplift distribution is assumed from reservoir head at the toe to tailwater at the heel.

However, the tensile stress is very small (0.03MPa). If zero tensile strength is assumed, and full uplift is assumed to penetrate to the crack tip, the calculations show that the crack would continue to propagate through the dam thickness. However, the dam was probably bonded to the foundation at least somewhat, and the foundation jointing probably interconnected enough that full uplift at a crack tip would not occur. It is unlikely that such a small tensile stress would result in complete instability, and undercutting of the dam toe during flood flows of the magnitude experienced (3.4m over the spillway crest). The weak bedding planes would daylight into the eroded hole with the loss of any keying effect from downstream rock layers. Assuming resistance only from the portion of the dam base in compression, a friction angle less than 50° would lead to sliding. It is likely that the friction angle of the weak bedding layers was considerably less than this, but there probably was some downstream passive rock, roughness, cohesion, and/or interlocking rock that prevented sliding until the reservoir reached the height at which failure occurred [33].

Teton Dam

Teton Dam, an embankment dam, whose maximum section rose 94m above the original valley floor and 123.6m above the lowest point in the foundation was located three miles northeast of New Dale, Idaho. The dam and its reservoir were the principal features of a multipurpose project, to serve flood control, power generation, recreation, and supplemental irrigation water supply for 40,000 hectares of farmland. When complete, the dam would form a 27.4km long reservoir with capacity of 356 million cubic meters when filled to the top. During the investigations, eight alternate sites within about 16km of the selected site were investigated, between 1961 and 1970. Construction works of the dam commenced in June 1972 and it was completed, first filling started in November 1975, but the dam failed on June 5, 1976 when the reservoir was at el. 5301.7 foot above sea level, 3.3ft below the spillway sill. The dam was a central-core zoned earth fill structure, refer Figures 22 and Figure 23. The crest elevation before camber was 5,332ft, and its length was about 3,100 ft. The volume of the embankment was about 10 million cu yds.

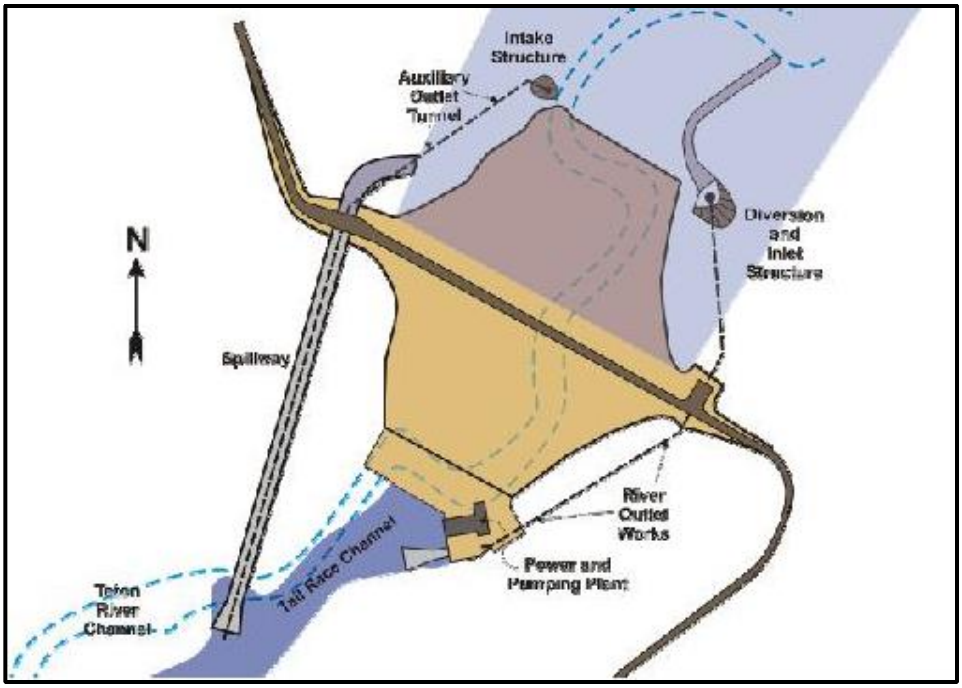


Figure 22: Plan view of Teton Dam [34].

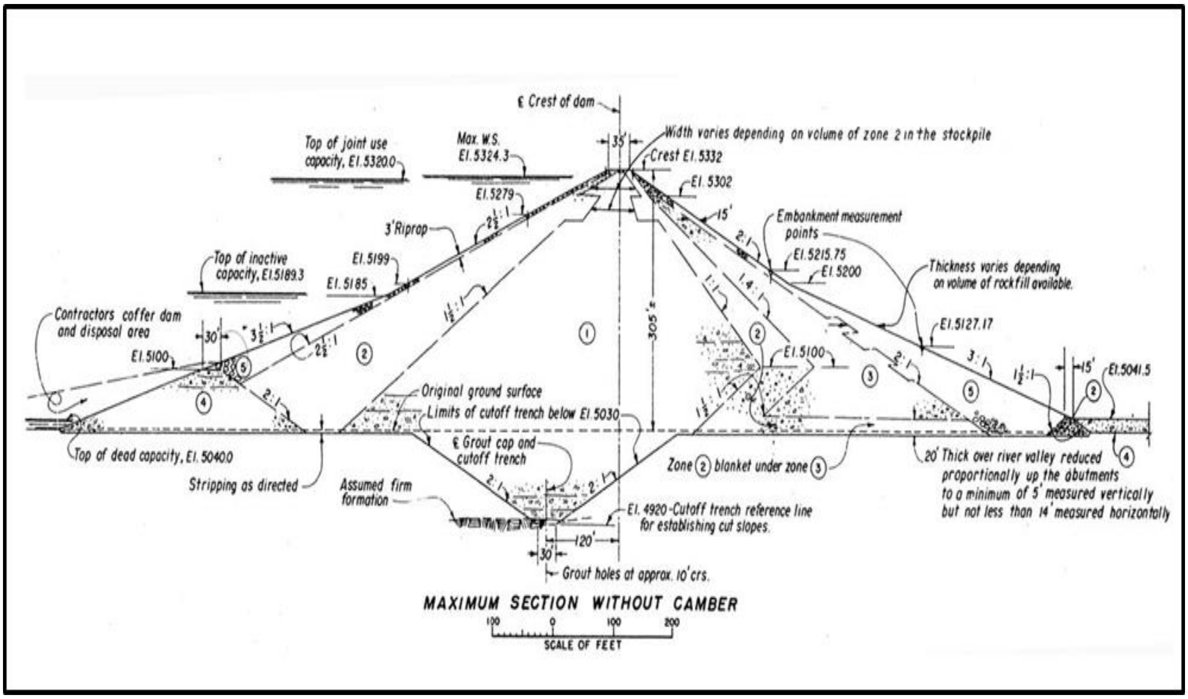

℄ Crest of dam
35'
Width varies depending on volume of zone 2 in the stockpile
Crest El. 5332
El. 5302
Top of joint use capacity, El. 5320.0
Max. W.S. El. 5324.3
El. 5279
$2\frac{1}{2}:1$
15'
3' Riprap
Embankment measurement points
$2:1$
El. 5215.75
El. 5200
Top of inactive capacity, El. 5189.3
El. 5199
El. 5185
$1\frac{1}{2}:1$
1:1
1.4:1
Thickness varies depending on volume of rockfill available.
Contractors coffer dam and disposal area
30'
$3\frac{1}{2}:1$
$2\frac{1}{2}:1$
①
②
305'±
②
El. 5127.17
El. 5100
⑤
④
El. 5100
③
$2:1$
$3:1$
$1\frac{1}{2}:1$
15'
El. 5041.5
②
⑤
2:1
Original ground surface
Limits of cutoff trench below El. 5030
$1\frac{1}{2}:1$
Top of dead capacity, El. 5040.0
℄ Grout cap and cutoff trench
Zone ② blanket under zone ③
20' Thick over river valley reduced proportionally up the abutments to a minimum of 5' measured vertically but not less than 14' measured horizontally
④
Stripping as directed
Assumed firm formation
2:1
2:1
El. 4920-Cutoff trench reference line for establishing cut slopes.
120'
30'
Grout holes at approx. 10' crs.
MAXIMUM SECTION WITHOUT CAMBER
100 0 100 200
SCALE OF FEET

Figure 23: Design cross section of the dam at river valley [35].

First signs of failure of the dam were the appearance of small springs in the right abutment downstream from the toe of the dam on June 3, 1975. These springs were clear water and did not appear to be serious in nature. On Friday, June 4, 1976, the entire downstream face of the dam showed no sign of any problem. The previous day leaks discharge was about 20 gallons per minute (1.6 liter per second). As of the evening on June 4, seepage was observed from springs 1,400 to 2,000 feet below the toe of the dam in two groups with a reported total flow of 100gpm (6.3 liter per second) and from a small spring midway on the right side, between the toe of the dam and the spillway. On June 5, the leak was flowing at about 500 to 800 liters per second from rock in the right abutment. By 9:00 a.m. the flow had increased to 1,100 to 1,400 liters per second and seepage had been observed about 40 meters below the crest of the dam. At 11:00 a.m. a whirlpool was observed in the reservoir directly upstream from the dam indicating the formation of the sinkhole in the upstream face of the dam. Alarm was initiated for the inhabitants downstream from the dam at10:30 a.m. As the dam breached, an estimated 300 million cubic meters of water headed down the Upper Snake River Valley at its peak release, of an estimated 28,300m^3/sec. The 22.9m high flood wave swept the valley, and although downstream warnings are believed to have been timely, deaths of 14 persons and property damage estimated variously from 400 million to one billion dollars have been attributed to the failure. Buildings and large areas of cropland were destroyed along with livestock and flooding the towns in its path, including Wilford, Sugar City, Rexburg, and Roberts. More than 200 families were homeless [35], [36].

The locations of seepage springs are indicated by red color dots in Figure 24 indicate the locations of the seepage springs, the uppermost red spot indicates the location of the sinkhole. The section shown in Figure 25 is along the approximate path of failure.

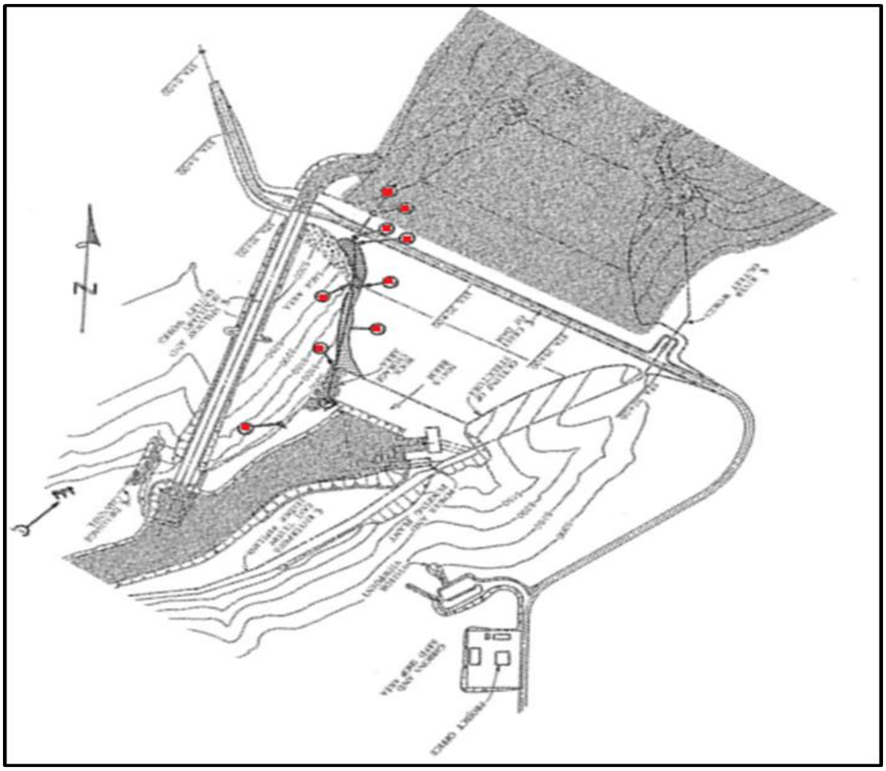


Figure 24: Location of springs on right abutment. Location of sinkhole at the uppermost top.

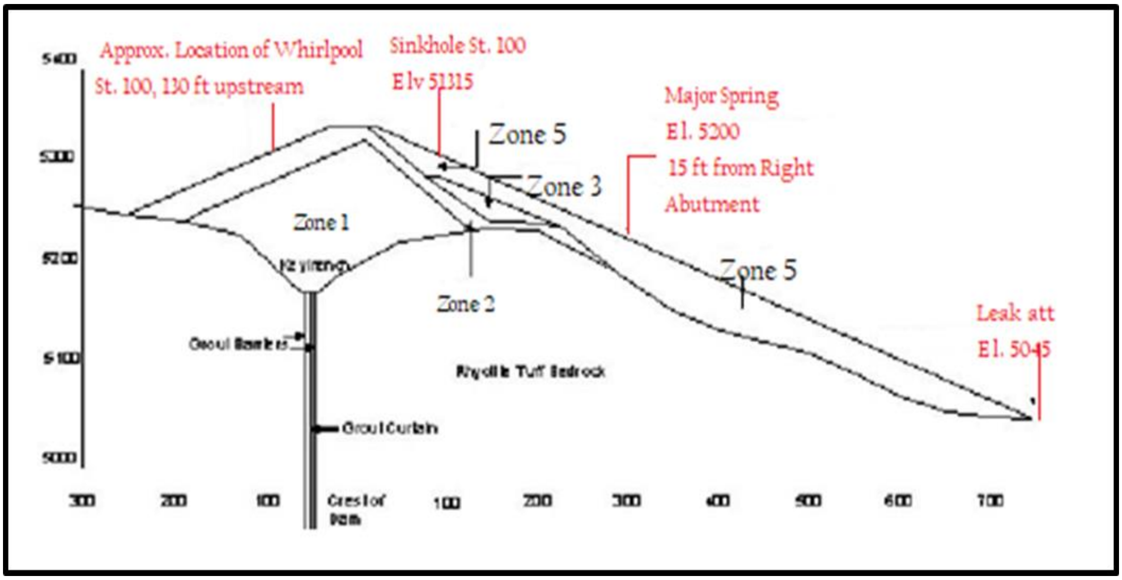


Figure 25: Section along approximate path of failure [37].

Although the dam mode of failure was piping leading to internal erosion in the dam body, as confirmed by all technical writings on this matter, it is unanimously agreed that the geology of the site and foundation conditions played a decisive role in initiating the failure. The fundamental cause of failure may be regarded; therefore, as a combination of geological factors and design decisions that, taken together, permitted the failure to develop. The embankment itself consisted of five main zones. Zone 1 was the impervious center core, which formed the water barrier of the dam. Zone 2 overlaid Zone 1 and extended downstream to provide a layer to control seepage through the foundation. Zone 3 was downstream, and its main function was to provide structural stability. Zone 4 consisted of the storage areas downstream from the control structure, and the temporary enclosures built to permit the work to be done. Finally, Zone 5 was the rockfill in the outer parts of the embankment. Some of these features are shown in Figure 23. The design of the foundation consisted of four basic elements:

1. 21m deep, steep-sided key trenches on the abutments above the elevation of 1,550m.
2. A cutoff trench to rock below the elevation of 1,550m.
3. A continuous grout curtain along the entire foundation.
4. The excavation of rock under the abutments.

These elements for the foundation were important because the types of rocks located within this area, basalt and rhyolite, are not generally considered acceptable for structural foundations.

At the dam site, the Teton River occupies a steep-walled canyon incised in rhyolitic ash flow tuff, a hard rock derived from distant volcanoes now long extinct. Extensive joints are common in this rock and are particularly numerous near the surface of the abutments. A cross-section of the canyon approximately along the longitudinal axis is shown in Figure 26 [38], [39]. Figure 27 shows the same long section but in distorted scale for clarify of terms [34].The walls of the canyon consist of later Tertiary rhyolite welded tuff, which is strongly jointed, with joint widths varying at different elevations typically between ¼ and 3 inches but with occasional joints up to 12 inches wide. Alluvium was deposited on the river channel to a depth of about 100ft. (30m) and the high lands near the ends of the dam are covered with an aeolian silt deposit up to about 30ft (9m) thick. The primary features of the site are the extensive joint system in the rhyolite-tuff, which makes it extremely permeable, and the abundance of the wind-blown silt deposit, which led the designers to use substantial quantities of this material in the dam cross-section.

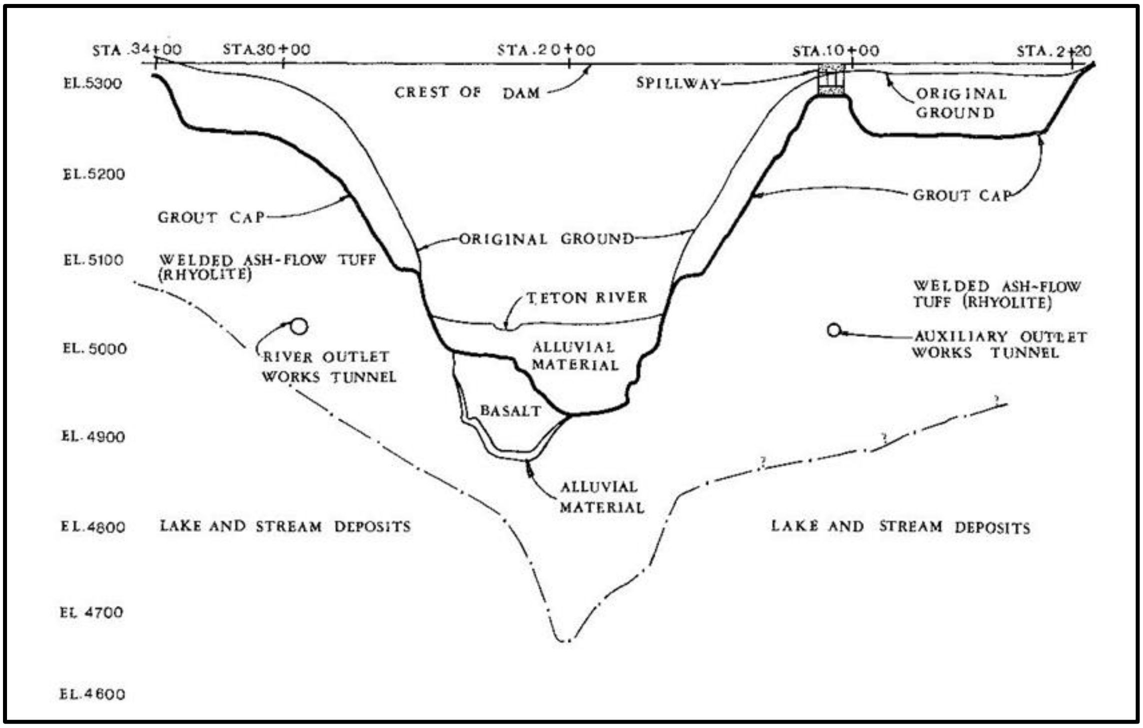


Figure 26: Cross-section of the canyon approximately along the longitudinal axis (showing Grout Cap) [38, 39].

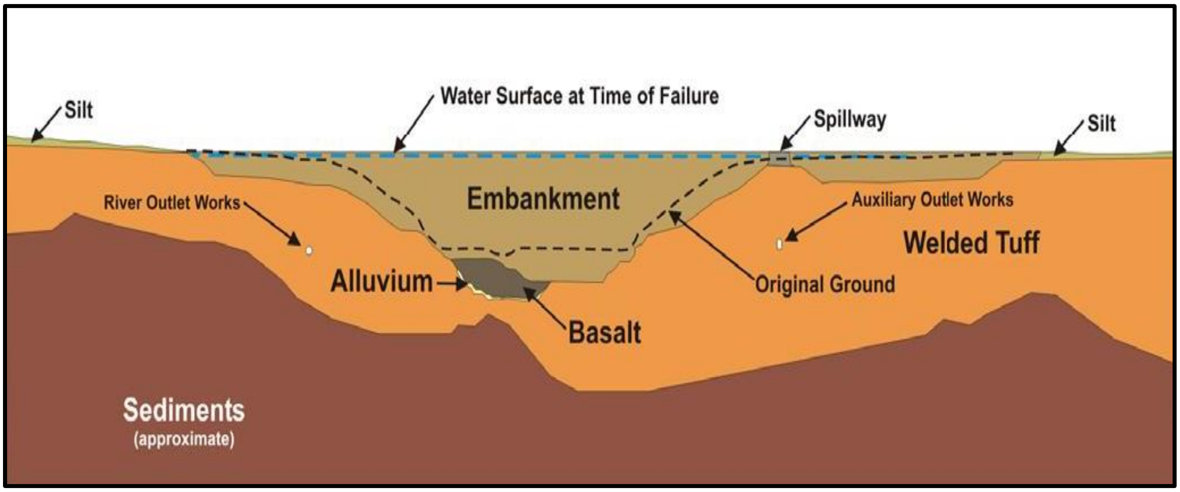


Figure 27: Cross-section of the canyon approximately along the longitudinal axis showing foundation materials (scale distorted) [34].

Extensive site exploration was performed prior to construction. Percolation tests and pumping tests revealed that the joints were capable of transmitting volumes of water over 100 gallons/min. These investigations indicated the presence of an extensive interconnecting system of joints, which made the rock extremely permeable and indicated the need to seal the joints in order to reduce the leakage to acceptable quantities. In order to investigate the possibility of sealing the upper foundation rock by grouting, an extensive pilot-grouting program was conducted on the left abutment. After the pilot grouting, the area was core-drilled, and water pressure tested. Since the amount of grout needed was a tremendous amount, it was concluded that it would be more economical to remove the top 70ft. rock in the abutments above El. 5100ft and incorporate a deep key trench to prevent seepage. Continuous high-angle joints in the right abutment have been traced for lengths of as much as 200 feet, but most are between 20 and 100 feet long. The aperture of most high-angle joints is less than one-half inch, but many joints are as much as several inches wide and some are several feet wide. Examples of the jointing can be seen in Figures 28 and Figure 29.

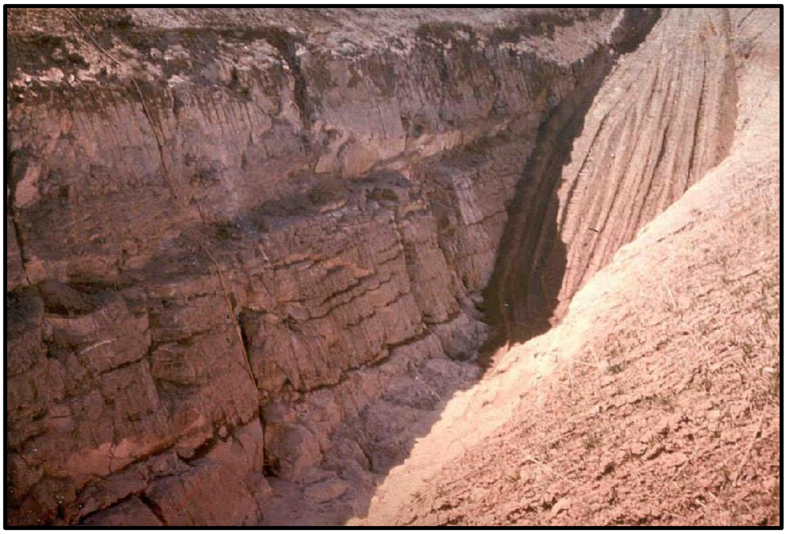


Figure 28: A view of the right abutment key trench showing the jointed nature of the foundation [34].

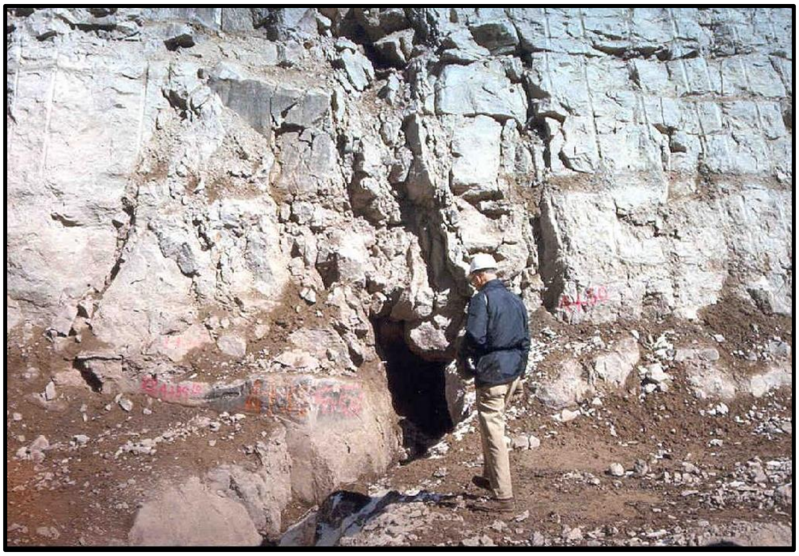
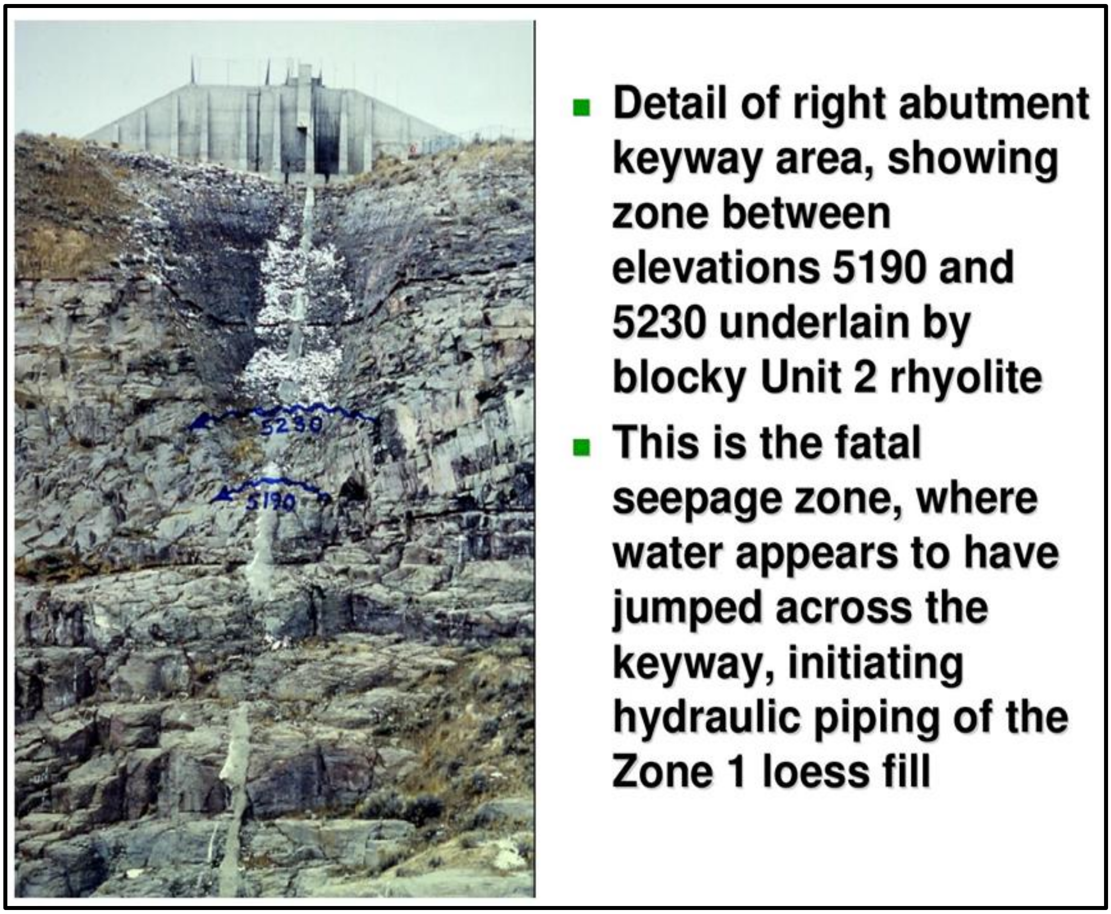


Figure 29: A view of a Large fissure in the right abutment key trench [34].

Post failure investigations, after the key trench on the right abutment was uncovered, showed exactly the fatal seepage zone location between el. 5190 and el. 5230 where seepage water jumped across the key trench overpassing the grout cap and initiating piping failure in the Zone 1 losses fill (refer back to Figure 25). This key trench is photographed and presented in Figure 30 on which the seepage zone is marked indicating the blocky unit 2 rhyolite beneath it.



- **Detail of right abutment keyway area, showing zone between elevations 5190 and 5230 underlain by blocky Unit 2 rhyolite**
- **This is the fatal seepage zone, where water appears to have jumped across the keyway, initiating hydraulic piping of the Zone 1 loess fill**

Figure 30: Photograph of the key trench showing the seepage zone [40].

Investigating the failure mode of Teton Dam, it may be best summarized in the words of the Internal Panel Report as follows:

1. The dam failed by internal erosion (piping) of the core of the dam deep in the right foundation key trench, with the eroded soil particles, finding exits through the channels in and along the interface of the dam with the highly pervious abutment rock and talus, to point at the right groin of the dam.
2. The exit avenues were destroyed and removed by the outrush of reservoir water. Openings existed through inadequately sealed rock joints, and may have developed through cracks in the core zone of the key trench.
3. Once started, piping progressed rapidly through the main body of the dam and quickly led to complete failure.
4. The design of the dam did not adequately take into account the foundation conditions and the characteristics of the soil used for filling the key trench.

The report adds that "the fundamental cause of failure may be regarded as a combination of geological factors and design decisions that, taken together, permitted the failure to develop." The Panel further described the geological factors as numerous open joints in the abutment and scarcity of better fill material than the highly erodible soil. The design decisions were "complete dependence for seepage control on a combination of deep key trenches filled with windblown soils and a grout curtain," a geometry of the key trench that "encouraged arching, cracking, and hydraulic fracturing" in the backfill, using compaction of the fill as the only protection against piping and erosion, and failure to provide for the inevitable seepage. In another place of the report stresses that the designers failed to understand and incorporate actual geologic information into the design, and geologists failed to present any strong resistance against the misinterpretation of this data. The fundamental cause of failure may be regarded as a combination of geological factors and design decisions that, taken together, permitted the failure to develop [35]. Another source describes the possible modes of failure by saying that the failure most likely began developing well before it was visible. Once visible, failure progressed too far for any possibility of repair. There are several possible modes and contributing factors to the failure.
These include:

1. Piping due to cracking of Zone 1 material. Post failure test data from left embankment soil indicated that differential settlement could have occurred. This could have been caused by any or all of the previously mentioned construction deficiencies. Hydraulic fracture due to arching could have occurred in the deep key trenches due to low lateral stress. However, the US Department of the Interior investigation concluded that neither of these modes of failure was likely.
2. Flow through the grout curtain. Due to the fractured nature of the foundation rock, it is impractical to expect that the grout curtain was tight. This could have allowed water flow into Zone 1 core, again, resulting in internal erosion.
3. Piping. Due to the fracture rock base, water could have entered Zone 1 through ungrouted fissures. Due to a lower hydraulic pressure on the downstream side of the dam, water would naturally move in that direction. This could lead to piping. Due to construction deficiencies above, all three of these piping scenarios were possible [41].

The failure of this dam has many lessons to be learned, which may be divided into the following categories:

Technical Aspects:

The failure of the Teton Dam could have been avoided as early investigations into the geology of the site showed that the rocks in the area were almost completely of volcanic origin. These volcanic rocks consisted of basalt and rhyolite. The geological survey report of the project of January 1971 described the rhyolite as "lightly to locally highly fractured and jointed, relatively light weight". This was also the condition for other possible sites located upstream of the site where the dam was constructed. These materials are usually avoided due to a history of erosion and deposition. The reason for the extensive foundation was the poor quality of the underlying material, including the grout curtain. The grout curtain, however, failed to do its job of preventing these materials from being easily washed away;, moreover the design did not provide for downstream defense against cracking or leakage, and did not ensure sealing of the upper part of the rock under the grout cap. The grout curtain was not constructed in three rows, and the reliance on a single curtain was judged to be "unduly optimistic." The dam and foundation were not instrumented sufficiently to warn of changing conditions.

Professional and Procedural Aspects:

At the first sign of a problem, the people at the dam site informed the Bureau of Reclamation. The bureau did not immediately inform the public due to fear of panic and because there were initially no signs of imminent danger, but the public was warned about 45 minutes before the collapse. It was determined that the people involved acted responsibly and were not punished for their involvement. However, the failure of the Teton Dam brought about changes in dam construction and operation by the Federal Bureau of Reclamation to ensure safety.

Educational Aspects.

This case demonstrates the importance of engineering geology and geotechnical engineering for civil engineering students. Engineering geology is important for evaluation of the suitability of foundation and borrow or fill materials. In this instance, both the rhyolite under the dam and the fine aeolian silt used as a fill material were deficient. Ironically, better borrow material was available in the valley downstream from the dam site, but the USBR decided that using it would be environmentally disruptive.

The failure of Teton Dam has made it clear that a dam site must have solid foundation material. An effective grout curtain may be an effective way of dealing with this unsuitable material, but only if there is a way to check if the grout curtain is an effective barrier once it is in place. Also, this failure showed that dams must be designed so that water level can be decreased in case of emergency. This was especially important to this case since the dam was allowed to begin filling while other parts (outlet structures) were still under construction. This failure also made each dam owner and operator, not only in the United States, but all over the world, aware of the lessons learned and apply them as procedures for higher level of dam safety, whether during investigation phase, design and construction phase and during first filling of the reservoir.

7. Conclusion

1. Geology plays an important role in deciding the safety of any dam and ensures its full usefulness in the future. Geological hazards, if bypassed during the investigation's phases, may lead either to failure later on, or costly repairs, or even the abandonment of the project as may happen if the reservoir tightness is a problem and cannot hold stored water. The role of a geologist begins with the inception of the planning and study phases to avoid all these consequences and continues afterwards in the construction phase through the collective technical assignments of all the other specialists to build and operate the dam successfully.
2. Recognizing potential geological hazards within the reservoir area and the selected dam site requires good knowledge about these hazards which can vary between potential landslides in the reservoir's rim, presence of karsts, soluble rocks in the reservoir or dam foundations, hidden faults in the foundations and the presences of hazardous materials or other geological anomalies in the foundations.
3. Foundation conditions at any site are most important to dams' safety, as it affects structural stability and stiffness to limit deformations to within acceptable limits and maintains the dam's integrity under the conditions that exist or may be expected to develop over time, such as excessive seepage under the dam with the accompanying high uplift pressures.
4. Studying the behavior of existing dams under problematic geological conditions or learning from case histories of dams' failures and incidents, is an important part of the technical education of both the geologist and engineers. So, they may learn the lessons and avoid similar problems in the project under consideration. Many such case histories are presented in the preceding paragraphs to show the variety of such problems and help understand their nature. Such case histories, together with the designer own experience and his technical background help in the selection of the most appropriate type of dam suited for any particular geological condition, avoiding such safety hazards like excessive settlement, which can lead to loss of freeboard, or differential settlement, which may cause dangerous cracking. Generally speaking, gravity dams require sound rock foundation and except for low small dams, concrete dams are not built on unconsolidated deposits because of their low bearing strength. Larger dams constructed in whole or in part on unconsolidated deposits should without exception, be earth or rockfill dams with the capacity to adjust to settlement in the foundation materials. Rolled Compacted Dams fill (RCC) has recently gained popularity due to their adaptability to various geological conditions similar in this as the rockfill dams.
5. Foundation treatment, type and extent, is determined early in the geologic and design stages of any dam. This by itself constitutes a major cost item and an important factor for ensuring the success of the project and its safety. Recognizing any geological hazard in the foundation can help to great extent in selecting the type of treatment and its scope. Sometimes, such treatment is

limited to excavating unsuitable materials to reach sound rock, which may need only treatment with dental concrete to seal cracks, joints and fissures. In many other cases, where problems of excessive seepage in the foundations are anticipated then deep cutoff trench, with or without grout curtain or diaphragm cutoff may be necessary to limit hazards like toe material erosion of the dam, excessive uplift on the dam base impairing its stability or excessive loss of water.

6. In all of the many cases of dam failures and incidents that are presented in the preceding paragraphs, it has been made clear that observed geological hazards in any site, as varied as they may be, or the other potential geological hazards that may be inferred by the designers, are critical to the safe performance of dams. This also demonstrates the importance of engineering geology and geotechnical engineering to civil engineers in the design and construction of dams. Securing the services of competent geologist and hydrogeologist, at very early stage, is very important to eliminate or reduce to safe limits any geological hazard that can face the performance of the future dam. Pinpointing these hazards themselves, helps to eliminate such hazards by producing safe design, guides to prescribing proper specifications and proper following of the implementation. In all these tasks the geologist has to take part.

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Dam Safety Problems Related to Seepage

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Abstract

Dangerous occurrences affecting dams take multiple forms, but seepage caused cases are the most numerous. Some of the cases are related to the geology of the foundation and the magnitude and type of discontinuities in the rock mass of the dam. Other are mainly due to of construction material in earth fill dams. Seepage occurs in all earth fill dams regardless of its materials, and seepage water can daylight at the downstream face causing erosion, piping and sloughing and instability; unless certain measures are taken. Instability can be controlled mainly by adding, filter material zones at the contacts with the clay core, chimney filter drain at the downstream part of the dam, filter zone or bench at the toe together with the drainage blanket under the downstream part of the dam. Seepage within the dam is enhanced by cracks which may result from uneven settlement of the dam due to different elastic behavior of the foundation materials, hydraulic fracturing, and differential settlement of parts of the dam or due to ground shaking in earthquakes. Preferential seepage paths can develop in such cracks, especially if the fill material is dispersive or suffusive. Similarly, such paths may develop along the contact surfaces of conduits installed under dams as outlet structures due to the low degree of compaction as a result of narrow trench dimensions. Using properly designed filter and drainages can reduce seepage quantities and the erosive force which causes internal erosion. In dam's foundation grout curtains or other type of cutoffs can reduce the hydraulic head and hence uplift under the dam and hinder seepage. Drainage, however, remains as the most efficient method in controlling this uplift in artesian conditions under dams, especially under concrete gravity dams. Generally, such drainage may take the form of drainage blanket and use of filters material.

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1. Introduction

The success of any dam project and its safe and fruitful performance can be achieved only, by systematic step by step procedure following prescribed disciplines. This begins with site selection of a promising site that fulfills topographical and geological requirements. From the start, the planning and design phases follow an interactive procedure with interconnected phases in a logical way. Hydrological studies, no doubt will decide on the availability of water in the river, whether during floods or low water seasons, but only by using hydraulic flood routing and judging from the topography of the site total storage can be decided. Following this, the magnitude of sedimentation of the reservoir will decide on the, dead storage, live storage of the reservoir and the various operation water levels and the selection of the maximum design discharge of the spillway and other outlet structures. All these questions fall within the scope of hydraulic engineering and were discussed in a previous paper [1]. Designing the dam to answer other hydraulic matters will depend on the type of the dam, its material of construction and foundation conditions. These involve seepage through the body of the dam, under-seepage in the foundations, uplift pressure on the dam base …etc. Hydraulic problems may arise during the operation of a dam which can threaten its safety. Such hazardous problems that need careful studies, and practical solutions are; seepage problems, scouring and cavitation induced by high velocity flow, sediments and debris blocking outlet facilities or even jamming of gates of outlet structures such as spillways are examples. An attempt is made in the following paragraphs to review anticipated or actual hydraulic problems related to seepage that are foreseen during design phase or appear later on during operation of dams, as these problems form one of the most common causes of dam failures and incidents.

2. Failure Statistics of Dams due to Hydraulic Problems

Well documented registers of dams' failures and incidents have been published by ICOLD. These registers show that during the last century, a rapid increase in large dams building occurred. From the given statistics, it can be seen that only about 5,000 dams had been built all over the world up to year 1950, but at the end of the 20th century, the number of large dams reached approximately 45,000 dams. The introduced strict safety rules and regulations combined with new technologies and innovations developed in the design and construction fields reduce the percentage of dam failures to just 0.5% nowadays from a figure of 2.2% before 1995. Nevertheless, each failure case can make big news in the media due to the large social, economic and environmental losses. Ideally, a zero percentage should be aimed at if possible.

The majority of existing dams today are embankment dams totaling 77% of all dams in the world, of these 64% are earth fill and 13% rockfill dams concrete dams and other type dams make up the remaining 32, refer Figure 1 [2].

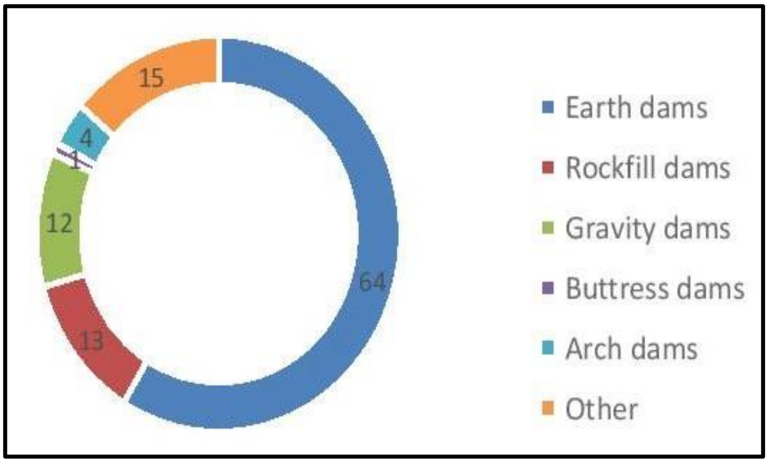
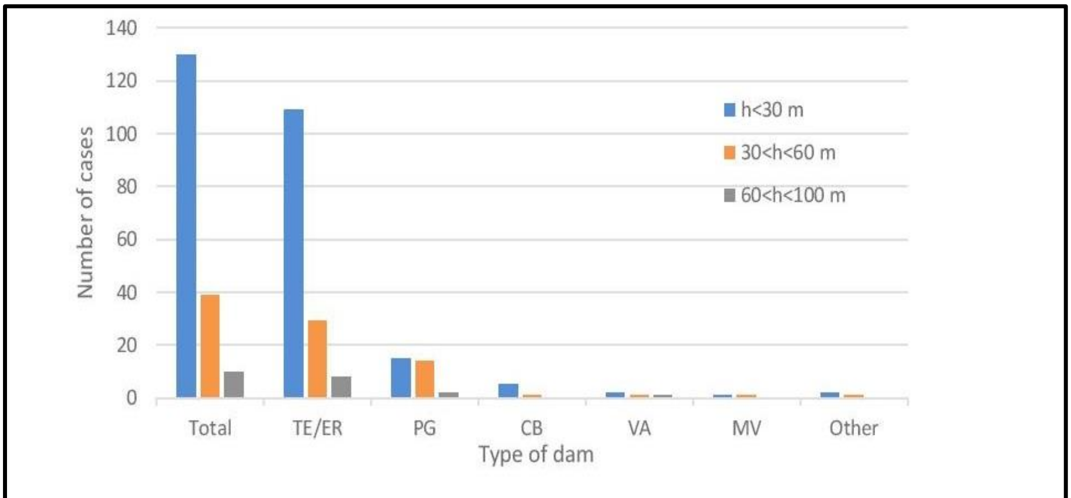


Figure 1: Percentage of all types of dams in the world, excluding China [2].

It follows that the number of failure cases in embankment dams hydraulic exceeds by far the failure cases in the other ones, which can be observed from Figure 2.

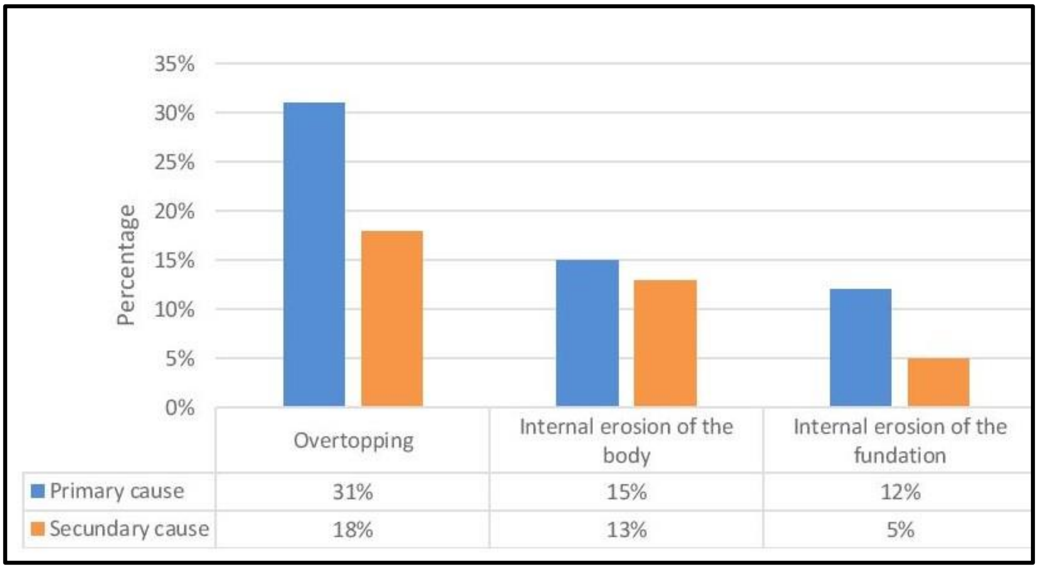


[TE – earthfill; ER – rockfill; PG – concrete gravity; CB – concrete buttress; VA – concrete arch; MV –multi-arch;]

Figure 2: Number of failures by types and height of dams, excluding China [3].

The bias in the above statistics towards embankment dams with 77% failures can be explained by the fact that the majority of dams constructed so far are embankment dams compared to concrete dams. Moreover, earth fill dams are more sensitive to hydraulic problems than the concrete dam due to the nature of their construction materials and the relatively weaker foundation on which they are founded.

The most common hydraulic problems that have caused dam accidents (failures and incidents) up to 1995 are shown in Figure 3. This is based on an (ICOLD) report published in 1995 under the title “Dam Failures-Statistical Analysis (Bulletin 99)”.



	Overtopping	Internal erosion of the body	Internal erosion of the fundation
■ Primary cause	31%	15%	12%
■ Secundary cause	18%	13%	5%

Figure 3: Causes of failures in embankment dams [3].

A study was conducted in 1998 by Foster, Fell and Spannagle from the University of New South Wales on Large embankment dam incidents up to 1986 and based on the (ICOLD) World Register, that includes a database of 11,192 dams; it showed that out of total 136 large dam failures, 121 cases are attributed to hydraulic problems. These dams cover a broad range of age, construction techniques, and foundation conditions. Table 1 shows statistics of embankment dam failures for different modes of failures. These statistics indicate that piping caused failures are the majority of all failure cases of embankment dams, and one-half of all piping failures are associated with conduits through the dam body or its foundations. Conduits are sources for discontinuities and are difficult to properly compact around. Moreover, nearly 50% of piping failures through the embankment have occurred during first filling and 64% within the first 5 years of operation. For piping through the foundation, nearly 25% of the failures occurred during first filling and 75% within the first 5 years of operation.

Table 1: Summary of embankment failures.

Mode of Failure	Number of cases		Percent failures (where known)	
	All failures	Failures in operation	All failures	Failures in operation
Overtopping	46	40	35.9	34.2
Spillway/gate	16	15	12.5	12.8
Subtotal	62	55	48.4	47.0
Piping through embankment	39	38	30.5	32.5
Piping through foundation	19	18	14.8	15.4
Piping from embankment into foundation	2	2	1.6	1.7
Subtotal	59	57	46.1	48.7
Slides	7	5	5.5	4.3
Earthquakes/ Liquefaction	2	2	1.6	1.7
Unknown	8	7		
Total number of failures	136	124		

First filling is considered the first true test of the embankment dam under hydraulic loading conditions. If a dam is poorly constructed or contains flaws and defects, first filling will likely expose these weaknesses. However, some piping failures can develop slowly over time (such as internal erosion into a karstic foundation). As indicated earlier, nearly 25% of failures occur after5 years of operation [4].

The main causes of failure in concrete dams, according to “Bulletin 99” ICOLD (1995) are summarized as shown in Figure 4 and Table 2 [5].

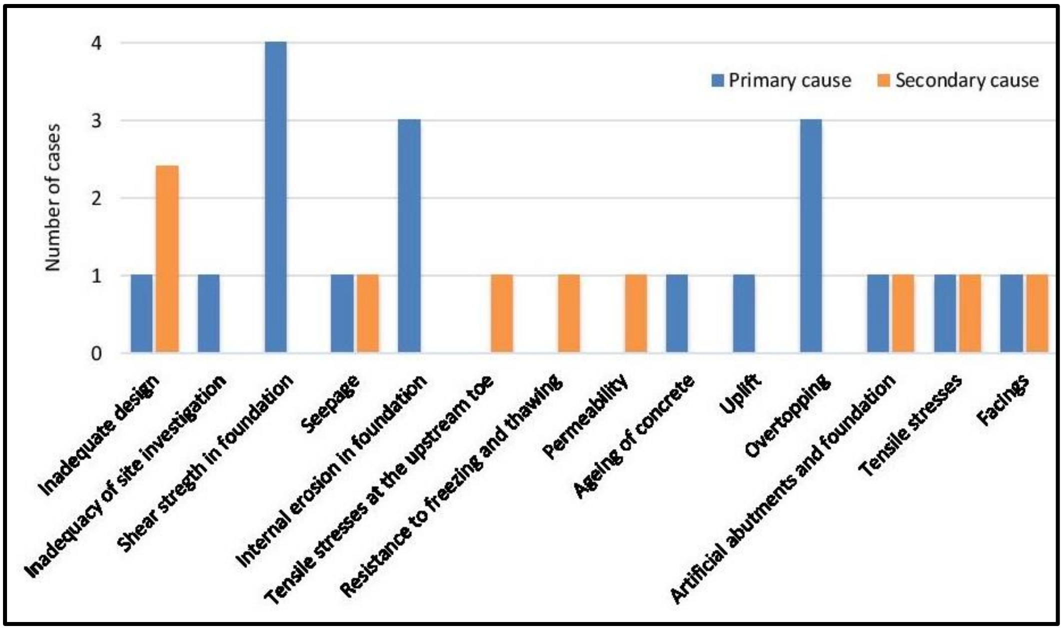


Figure 4: Main causes of failure of concrete dams.

Table 2: Main causes of failure of concrete dams.

	Details	Number of Cases	
		Primary Cause	Secondary Cause
Design	Inadequate design	1	14
Foundations	Inadequacy of site investigations	1	
	Shear strength of foundations	4	
	Seepage in Foundations	1	1
	Internal Erosion in Foundations	3	
	Tensile stresses at the upstream toe		1
	Resistance to Freezing and Thawing		1
	Uplift	1	
Dam	Permeability of Concrete		1
	Aging of Dam	1	
	Overtopping	3	
	Tensile Stresses	1	1
	Facing	1	1
Artificial abutments and foundation			

3. Embankment Dams: Hydraulic Seepage Problems

All embankment dams have some seepage, and many suffer from excessive seepage. As water in the reservoir seeks paths of least resistance through the dam body, and its foundation, it may pose some hazards on the dam safety. Excessive seepage may lead to dam failure if it is not treated and controlled properly. If seepage flow is allowed to continue unhindered in appreciable quantities, then the seepage force may erode fine soil particles and wash them out causing piping failure of the dam in internal erosion process or create uplift problems. Seepage, therefore, may be considered as one of the most common safety hazards for embankment dams, and many failures of such dams have been recorded in dam failures registries. The basic problem facing designers and operators is trying to discern how seepage is affecting a particular dam and what measures, if any, must be taken to ensure that the seepage does not develop and adversely affect the safety of the dam.

Protection against such hazard should be done in the design stage, and if it appears during the dam’s lifetime, the case must be investigated carefully, and the application of the necessary remedies should be done early to prevent it from developing into failure conditions. Seepage through the dam body can emerge anywhere on the downstream face, above the toe or on the downstream abutments, at elevations below the normal pool. In such a case the phreatic surface should be lowered in order to confine its exit point inside the dam body.

For the simplest case of a homogenous dam on an impermeable foundation, the picture of seepage through the dam body, without using any measure to lower the phreatic surface is as given in Figure 5.

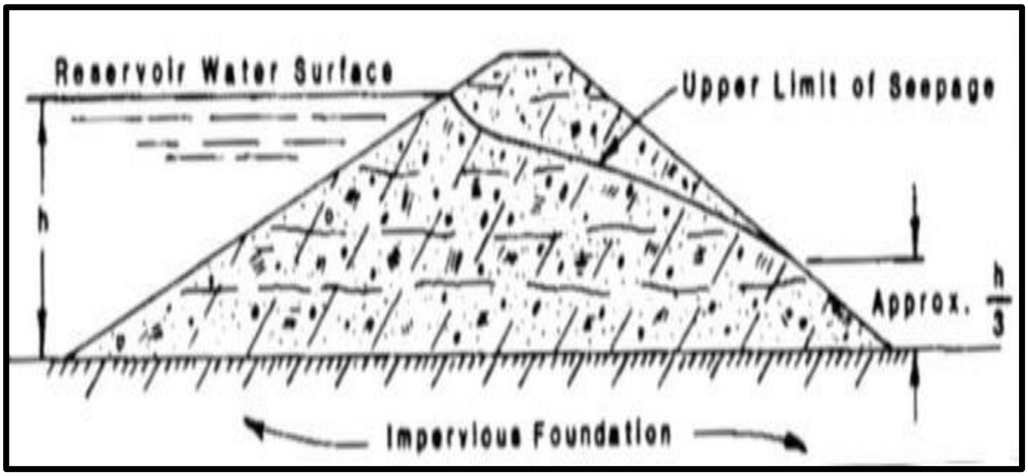


Figure 5: Homogenous dam showing phreatic surface of seepage.

In this type of dams, there is the danger of sloughing in the downstream slope and internal erosion leading to failure when the water level in the reservoir is high and pore pressure is also high. The same thing will happen as a result from the rapid draw of the reservoir and the upstream slope is saturated. Both slopes may have to be made very flat to avoid such conditions, or to add filters at the toe, or lay a drainage layer or drainage blanket within the base at the downstream, as shown in Figure 6 and Figure 7.

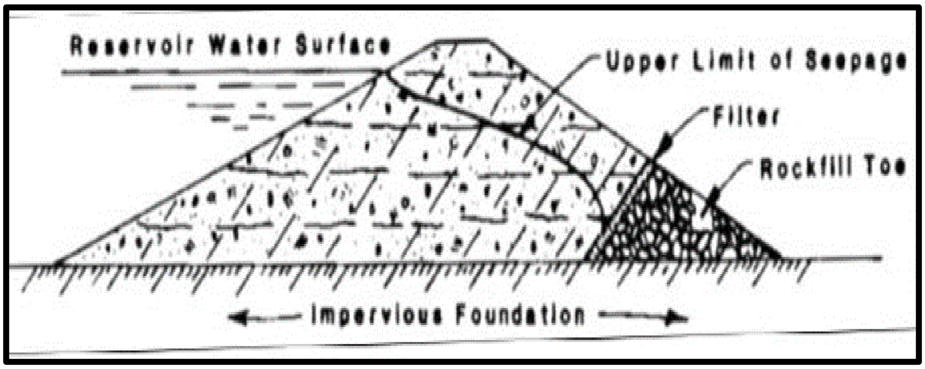


Figure 6: Using Rockfill toe and filters.

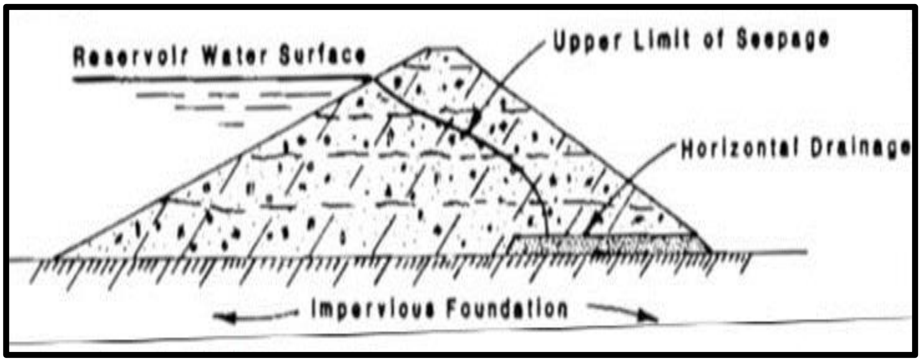


Figure 7: Using horizontal drainage blanket.

The last alternative is to add chimney drain within the dam to intercept the phreatic surface inside its body, and relieve seepage water afterwards from the downstream in a safe way as in Figure 8.

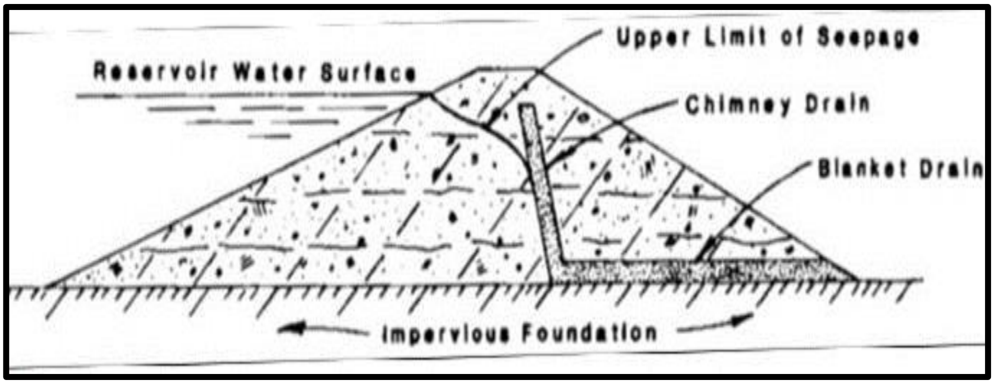


Figure 8: Using chimney drain.

Homogenous embankments may be used for low embankments, but not for high dams as safety hazards of dams' increase with increasing height and storage. As far as seepage is concerned, the current design procedures call for minimizing seepage and limiting its hazards instead of eliminating it completely, which may prove in most cases to be very costly or even impossible.

The need for constructing large dams has led to development of zoned fill dams; where different types of soil materials may be used and placed in such a way to protect from seepage. First of all, an impermeable core in the middle, which can be of clay, or a diaphragm (concrete or asphaltic concrete), will act as the main defense against seepage. On the upstream and downstream sides of the core, filter zones should be also designed and added to secure against any residual seepage or leaks through cracks in the core resulting from earthquakes, settlement or hydraulic fracturing. Seepage water is then collected and removed in one of the safe ways as indicated previously. It can be said, therefore, that the design philosophy has changed to controlling seepage and lowering the saturation of the phreatic surface and minimizing seepage quantities by incorporating filters and drainage elements, thus permitting steeper slopes and consequently higher dams. The extra safety drawn from using filters can also help in selecting semi impermeable soils and diaphragm for the core provided that the filter's arrangement and design fulfil the safe exit of the seepage water. One such case can be cited from the construction of Haditha Dam in Iraq, where dolomite fill is used for the central core with asphaltic concrete diaphragm, which is followed by the deep grout curtain in the foundation, refer to Figure 9 [6].

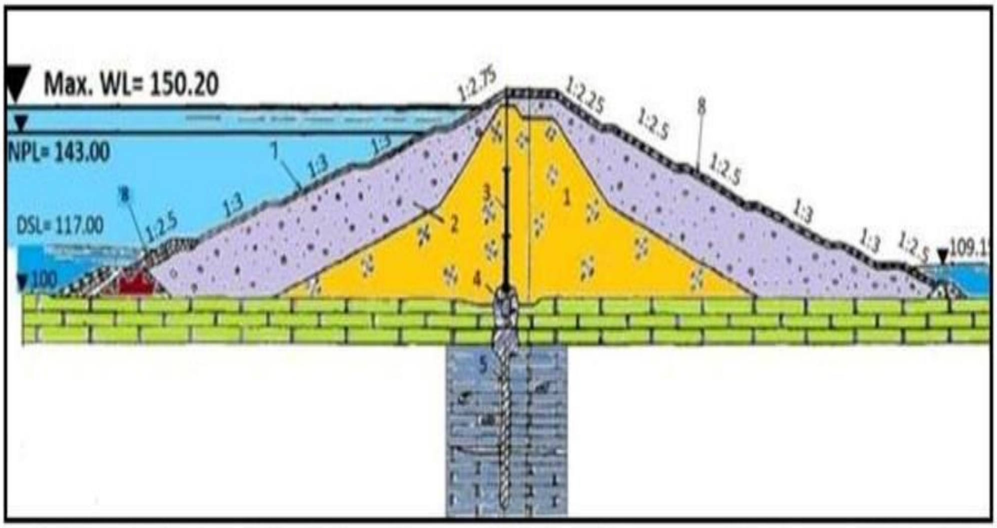


Figure 9: Cross section of Haditha Dam showing dolomite core and asphaltic concrete diaphragm.

In this Project, the permeability of the dolomite core varied between $1.1\text{x}10^{-3}$ and $3.1\text{x}10^{-5}$cm/sec which makes it permeable to semi-permeable. While permeability of the asphaltic concrete diaphragm is below $1.0\text{x}10^{-7}$cm/sec. Controlling seepage through the foundations of the dam, being an embankment or concrete dam, is important to prevent the foundation material from piping and washing away, which could result in structural failure due to loss of support. At the same time, it is important to control seepage in order to reduce uplift pressure.

Measures that can be taken vary according to each case from using impervious upstream blankets, and/or grout curtains or diaphragms and cutoff trenches under the dam itself. These measures will reduce uplift, lengthen the seepage path and reduce the exit gradient of the seepage water at the dam toe preventing the formation of boils, which are signs of internal erosion in the foundation material [7]. Seepage problems remain the same for both permeable foundations of silts or clays as for rock foundations, which contains solution channel, rock jointing and cavities. Figure 10 represents a case of under seepage of an earth fill dam on silt and silty sand foundation.

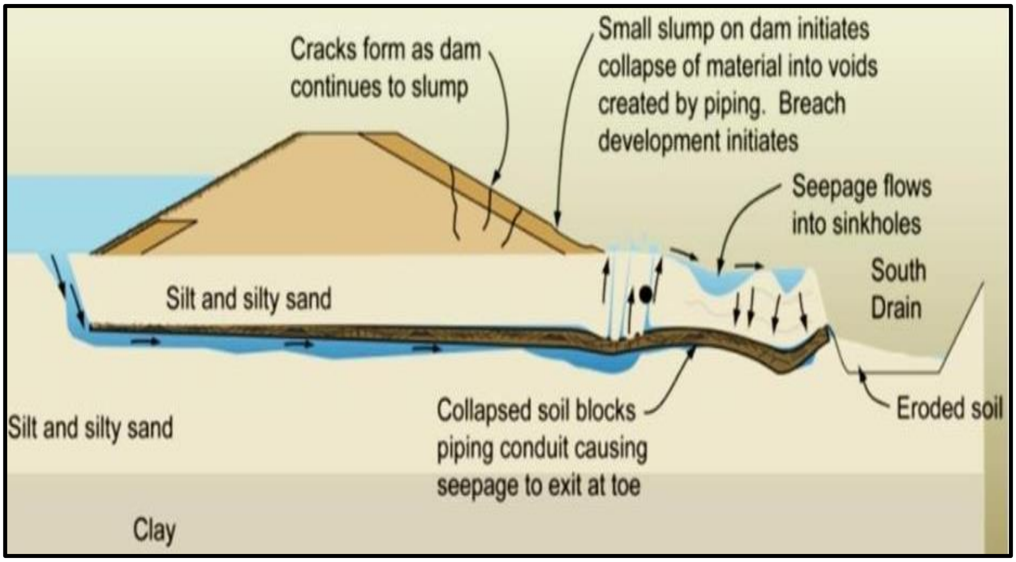

Cracks form as dam continues to slump
Small slump on dam initiates collapse of material into voids created by piping. Breach development initiates
Seepage flows into sinkholes
South Drain
Silt and silty sand
Silt and silty sand
Collapsed soil blocks piping conduit causing seepage to exit at toe
Eroded soil
Clay

Figure 10: Piping through dam foundations.

4. Concrete Dams: Hydraulic Problems as Consequence of Seepage

Seepage problems take special weight in the safety considerations of concrete dams. They are normally subject to various influences, which are related to their safe operation and existence such as; deformation of the foundation, strength of their materials, stability conditions, and aging. Seepage, however, plays a special role in unison with all these factors exasperating them in addition to its own negative role. The adverse effects of seepage on concrete dam safety are mainly from the uplift pressure on the dam base, which can be easily produced because of dam foundation seepage. The uplift pressure can decrease the anti-sliding force of the dam and threaten its stability. Moreover, concrete and masonry dams can be affected by the percolation of seepage water through the dam body in one or more ways. It may percolate through cracks in the dam itself which can cause leaching of its material and creates weakness planes. The historical accident of Bouzey Dam in France was directly related to this phenomenon. Seepage through the foundation can result in internal erosion of its material in addition to affecting the sliding stability of the dam. One research paper lists three sources of negative impacts, which can result from seepage, namely:

1) Dam seepage pressure shown as uplift pressure in dam foundation and seepage pressure within the dam body itself. Both are caused by seepage water and have certain influences on dam stability, deformation, and stress.
2) Dam leakage causes the water to take the fine particles out of the dam body and form a seepage passage, which endangers the stability of the dam.
3) Upstream impoundment cannot only seep through the dam body and foundation, but also seep downward around the bank slopes at both ends of the dam [8].

The case of Bouzey Dam, which failed on Saturday 27th April 1885, is a good example of more than one factor of those mentioned above acting together. The dam was a straight masonry gravity structure with a length of 528 meters which was built on the L'Avière River, a tributary of Moselle River near Epinal in Eastern France. When the dam failed, the ensuing flood-wave in the valley of the L'Aviere River resulted in drowning 85 people and causing extensive damage on canal works, railway structures, bridges, villages and farms in the downstream.

The original design was to have a height of about 20 meters with a crest thickness of about 4 meters. This thickness was to be carried down to a level about 4.3 meters below the top of the dam where the downstream face was to begin a concave circular curve extending down to the toe resulting in base thickness of about 11.3 meters. Construction was started in 1878, subsequently, a decision was carried out to increase the height about 2 meters without altering the other dimensions of the dam disregarding the "middle third principle" which states that on any horizontal plane within a gravity dam, the resultant of forces should act within the middle third. This is to ensure that the stress would be compressive at all points on that plane otherwise tensile cracks may be formed. Late in 1881, the work was completed, and the impoundment was started. Leakage soon appeared, totaling approximately 57 liters per second. About a year later, two cracks were discovered in the dam. As a safeguard, the allowable operational levels of the reservoir were lowered. On March 14, 1884, with the reservoir level about 2.7 meters below the maximum, a 137-meter long section of the mass slipped abruptly on its base and moved downstream as much as 380 millimeters. This was accompanied by a rapid increase in leakage to about 108 liters per second, but no remedial measures were taken. In the fall of 1885, about 1.5 years after this mishap, the reservoir was drained and many cracks in the upstream face were discovered, with one break extending 91 meters horizontally. Evidently, cracking had also severed the dam from its cutoff wall, which extended into the foundation at the heel. This crack at the base was then covered by a longitudinal block of masonry, which in turn was sealed with puddled clay. The downstream toe was extended outward and downward by addition of a masonry mass which roughly doubled the dam's base thickness. It gave a flatter slope to the lower downstream face, keying into the original mass at about mid height. The cracks in the old masonry were grouted, and drains were installed. The remedial program was finished in September 1889, Figure 11.

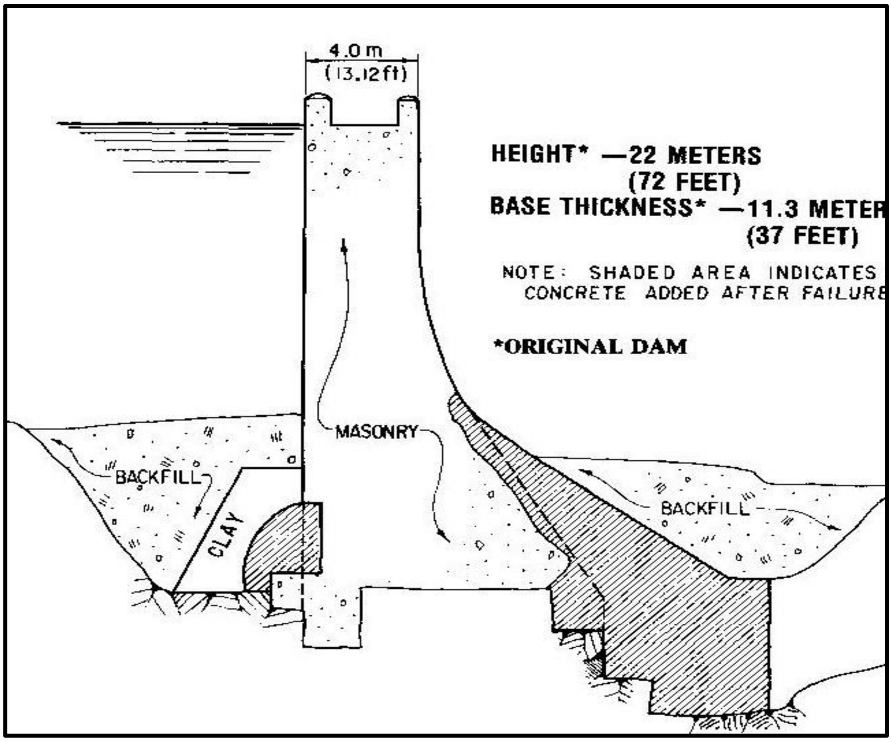


Figure 11: Bouzey Dam profile after modification and remedial program.

In November 1889, reservoir impoundment was begun. When the water level reached maximum, the crest deflection was nearly 25 millimeters at some points. On April 27, 1895, the entire top part of the dam - a mass roughly 10 meters high and 183 meters long - broke away and loosed a torrent of water upon the village of Bouzey. Continuing its rush down the valley of L'Avière River, the flood left several other villages in ruins. Among factors related to the failure, water pressures inside and under the dam were undoubtedly contributory. The sandstone foundation was cracked, and the cutoff at the heel was not extended adequately into the rock. Both of these conditions could allow water under the structure. Faulty masonry joints could permit similar leakage at higher elevations. Detrimental hydrostatic pressures, therefore, could have developed easily. Such pressures, if applied in the upper parts of the dam where structural thickness was marginal, may have been a primary source of trouble. Yet the weakening of the dam appeared to be progressive. Failure did not occur as soon as the reservoir was full, but instead, many months passed before the signs of distress were discovered. After the repairs had been completed in 1889, an even longer period ensued before the collapse, even though the storage levels were kept high. Investigators of the disaster wondered whether other, more subtle, forces of destruction were at work. In the search for a cause of failure, the mortar used to bond the masonry was an outstanding suspect. There is apparently no question that its quality was inferior. If the preparation of lime mortar was done carelessly, as was alleged in this case, some of the lime may have remained unslaked or only partially slaked. If incorporated into a structure in this unstable state and then exposed to wetting, the lime will complete its slaking and tend to expand in the process. Used in Bouzey Dam, this could have resulted in weak mortar joints constituting potential planes of separation, which were good conduits for seepage. Whether the cracking in the dam followed such planes is not clearly evident.

Various possible causative factors may have acted together. The comparatively thin masonry section would have been susceptible to tilting away from the thrust of the reservoir water, accompanied by opening of cracks at the base and in the masonry and by gradual deterioration of mortar. Even acknowledging that the design dimensions were marginal, including the lastminute increase in height, one unavoidable conclusion is that defective materials and poor workmanship deserve much of the blame for the Bouzey Dam disaster [9].

One case which highlights the problems created by seepage water seeping into concrete dam body and result in costly remedial works, is that of Ajba Dam. This dam is part of the hydro-power plant Plave on the Soča River in Slovenia, completed in 1940. The dam has structural height of 39m, and crest length of 72m. It may be classified as a combination of a barrage and gravity dam. The retaining structure consists of four massive piers with three large gates. The piers are connected at elevation 111.5m a.s.l. with bridging elements in pairs, one at the upstream and one at the downstream side of the piers. The small bridges are arched shaped and made of reinforced concrete. The foundation level is 92.1m a.s.l. and the roofing structure with the mechanism for opening and closing the gates is at the 120.0m a.s.l. The roofing structure or ‘upper deck’ is constructed on top of the four extended piers, and it is made of arching girders, fixed with cross girders and reinforced concrete plate. The usual water level is at 106m a.s.l, refer to Figure 12.

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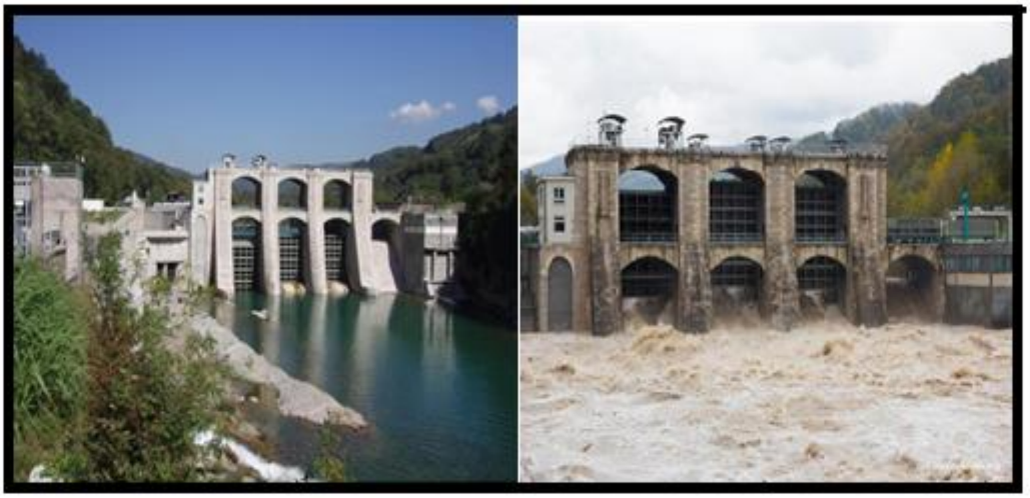


Figure 12: Views of Ajba Dam (212) [10].

Sixty-five years of exploitation caused extensive damage on concrete: e.g. cracks, corrosion of reinforcement, damages of construction joints and stone blocks, deterioration due to freezing and thawing cycles … etc. In the concrete elements of the dam most of the damage was caused by the degradation effect of frost cycles of seepage water penetrating through faulty construction joints and cracks. These damages were very severe at locations where constant leakage and wetting of concrete had occurred. Open construction joints and numerous cracks were clearly visible on all sections of the dam. Most of them were initially caused at the time of construction due to the technology used in construction. At these cracks, leakage was observed where the width of cracks was up to 3mm. It was necessary to seal the open joints and cracks to stop seepage water and re-establish integrity of each construction elements and prevent further degradation of the concrete and the reinforcement.

Based on the findings of the visual inspection and laboratory tests, the scope and cause of damages were established, and an extensive program of rehabilitation was then carried out. Among the various applied measures; sealing all cracks was foremost, using polymer cement mortars applied manually. On large cracked sections, the mortar was applied using shotcrete, and new protective layer of concrete was applied over the exposed reinforcement bars. Concrete surfaces were smoothed with fine-polymerized cement mortar. Protective coating of polymerized cement was applied; waterproofing layer was carefully placed.

Five years after rehabilitation works were finished most of the former cracks and joints are still closed and not visible. At some locations, especially where leakage was not entirely prevented, the cracks and flowstone deposits reappeared [10].

Early experience with Rolled Compacted Concrete dams construction involved problem of seepage water leaking from the dam body, especially at the horizontal joints between lifts, which caused alarm. One of these early cases was that of the Willow Creek Dam located on Willow Creek in at Oregon in USA east of the city of Heppner. It was the first major dam in the United States constructed of roller-compacted-concrete. The dam's original purpose was primarily to store water for flood control, but also to serve recreation, fishing and wildlife and irrigation uses. The reservoir level can be a maximum of 644.2m a.s.l. and a minimum of 628.8m a.s.l. for a total usable storage capacity of 12 x 10^6m^3. Flooding was common for this stream, and a major flood had killed 247 people of Heppner's population in June 1903, which had resulted from a thunderstorm and known as the Heppner Flood of 1903. The observed Creek flow was then 1,000m^3/s.

The dam was constructed by the US Army Corps of Engineers between November 1981 and February 1983. Construction was completed nearly on schedule, despite the workers initially being unfamiliar with the materials, processing, and techniques. The cost of the dam was about $35 million, 330,000 cubic meters of concrete were finished in less than five months at about $17 per cubic meter which included additional efforts to correct defects.

As the reservoir began filling significant leakage was evident through the seams of the layers of concrete. The reservoir was drained, and a $2 million remedial effort included injecting grout through bores drilled from top to bottom. The initial rate of leakage was 33 cubic meters per second. After remediation, the leakage was less than 11 cubic meters per second. Concern over the dam's safety has continued ever since, especially with the memory of the 1903 flash flood [11]. Figure 13 shows a view of this dam.

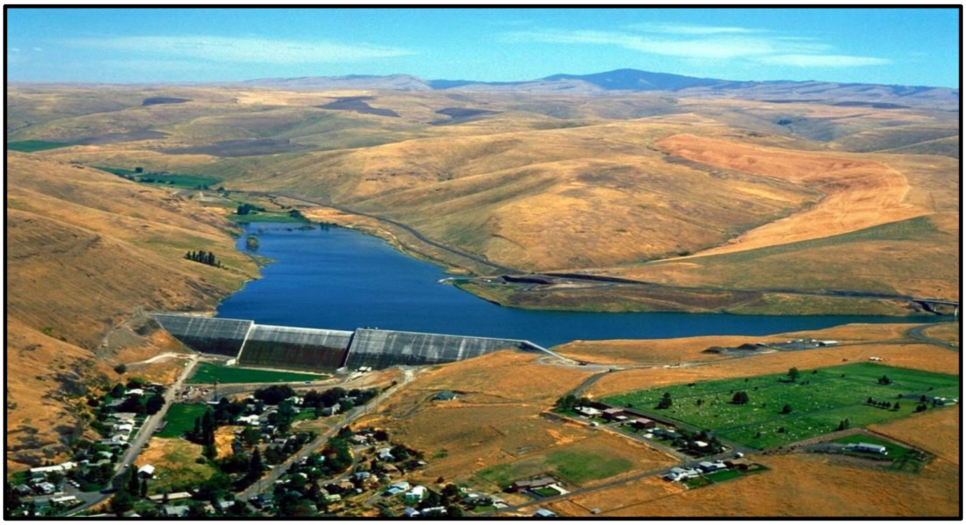


Figure 13: View of Willow Creek Dam [12].

Controversy over these dams has continued since its construction. Paradoxically, given the tragic loss of life in 1903; the initial proposal to build Willow Creek Dam was never popular here. The impoundment was regarded with suspicion. One of the reasons they say that the dam leaked like rusty bucket. Moreover, seepage frightened some and resulted in vegetation so dense on its concrete face that a local logger threatened to graze sheep there, which was actually done as seen from photographs, refer to Figure 14 [13]. In 1988, a Corps of Engineers lake scientist warned that water and chemicals from the nutrient-rich reservoir were dissolving the concrete as it seeps through. An inspection team later declared the dam safe, but the Corps said water action eats up to 20 metric tons per year of the dam's 900,000ton weight, though not enough to weaken it. Officials in Oregon consider seven dams across the state to be in “unsatisfactory" conditions. According to a statement made by Diana Enright, the spokeswoman for the Oregon Water Resources Department. Enright provided a list of seven troubled dams in Oregon; one of them was Willow Creek Dam. All seven Oregon dams deemed “unsatisfactory” for safety and high hazard dams.

While it’s hard to find out which dams have the potential to fail, a federal database lists those that would have catastrophic consequences if they did [15].

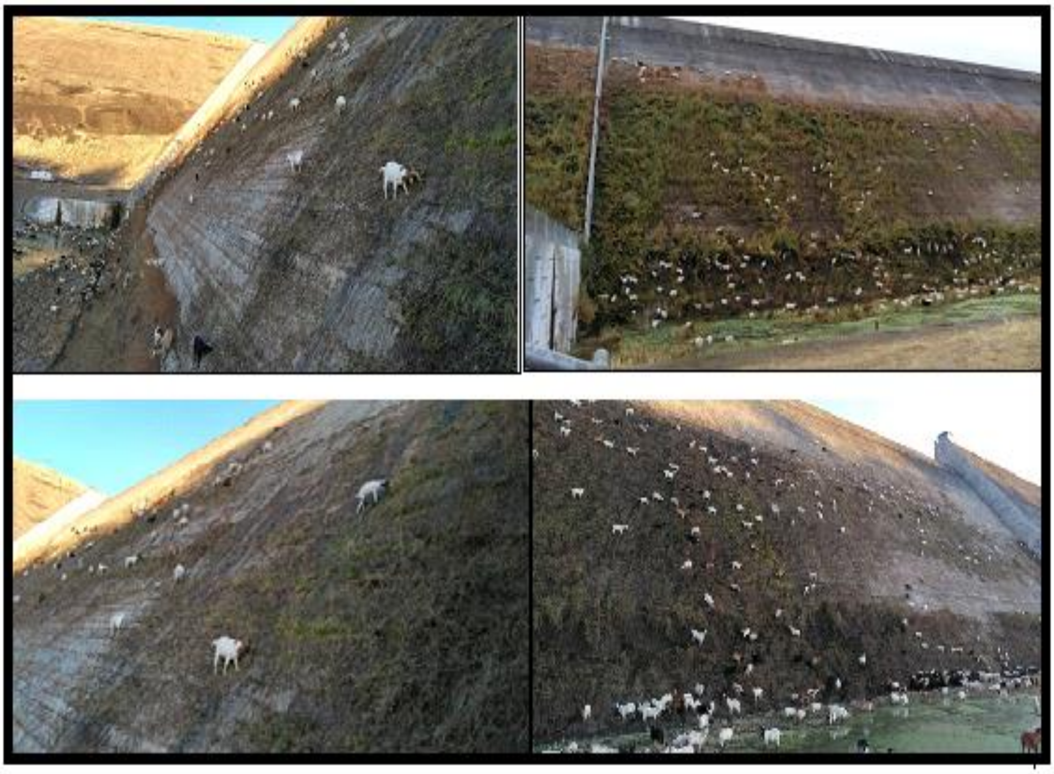


Figure 14: Leakage signs on the downstream side of the dam, and at the toe observe the heavy vegetation and the grazing goats [13].

5. Internal Erosion in Embankment Dams

Internal erosion is one of the most common modes of embankment dam failures, which is caused by extreme conditions of seepage. Inspecting the statistics shown in Figure 3, shows that the total number of all dam failures and incidents by the internal erosion amount to 40%, adding together failures in the foundation and body of dams. In order to give a good insight of this mode of failures, its causes and forms, elaboration may be necessary, and some details are presented in the following:

5.1 Definition

Internal erosion is one of the main causes of dam failures, and one of the most difficult potential failure mechanisms to predict. Normally, seepage occurs in every earth fill dam, but it is uncertain when the seepage occurs and which path it will take. Many of these paths may never be seen or detected during the life of an embankment. The real problem starts when the seepage begins to dislodge and move particles of soil from one location to another in what is called internal erosion. This can be of concern for the safety of an embankment dam, and at times, one concern for concrete dams and other concrete hydraulic structures founded on rock or soil. Many geotechnical problems; such as differential settlement, core cracking or hydraulic fracturing are associated with internal erosion, but difficult to identify with regards to critical seepage mechanisms within an embankment dam. Better understanding of this may best be developed by a thorough study of various case histories, which can give insight and understanding on the behavior of dams allowing the adoption of safeguards against them in existing or future dams. Internal erosion may come under different headings in dam engineering literature; It may be included under “Concentrated leak erosion”, “Scour”, “Seepage erosion”, “Piping”, “Backward erosion piping”, “Internal instability”, “Suffusion”, “Internal migration”, “Heave”, but all these terms refer basically to the same failure [16].

5.2 Causes and Modes of internal erosion

Concentrated leak erosion, or tractive force erosion, can occur when preferential flow paths develop in earth embankments, their foundations, or at contacts between the fill and concrete structures or bedrock leading to the subsequent negative results. The majority of catastrophic dam failures are due to concentrated leak internal erosion mechanism. In this mechanism, soil particles are dislodged and carried away by seepage water along a preferential flow path and end at the filter zones, if there is any, or another downstream exit.

In some cases, defects such as cracks or other flaws in an embankment or foundation might not directly result in concentrated seepage. It is possible that pore pressure, induced by direct pool head pressure may build up within, beneath, or upstream of an impervious layer or zone and can lead to forces capable of moving material from and breaching the impervious layer from hydraulic fracture or blowout. Once the impervious materials are breached, a concentrated leak could rapidly develop along a flaw or geologic defect that previously did not have sufficient velocity to dislodge soil particles. Potential flaws that can lead to concentrated leak erosion in embankment dams or foundations must be considered. Major causes of these defects are presented hereunder.

5.2.1 Differential Settlement

All embankment dams will settle to some degree during and after construction. This settlement may not be uniform, and differential settlement should be expected in every dam. When the foundation profile is irregular and the depth of fill varies, differential settlement will occur. When differential settlement is large enough, cracks can form in areas of tensile stress. Typically, these will be transverse cracks that could potentially be continuous across the core and are therefore, of most concerns. Longitudinal cracks may develop due to differing foundation conditions beneath the core and shells and/or due to differences in elastic moduli of the various zones. Longitudinal cracks are generally of lesser concern but could potentially allow reservoir water to access two separate transverse cracks that would not otherwise be continuous from upstream to downstream. Additionally, dams founded on collapsible soil deposits can settle after the reservoir fills. If wetting occurs during first filling, or during a flood event for flood control impoundments, the settlement can be large and dramatic. Earthquakes may be another cause for cracking of the dams resulting from ground shaking and differential settlement that follows. These cracks are not different in their impacts on dams as any other cracks, and if not treated in good time, they may develop into good conduits for seepage water leading to internal erosion. A good description of cracking in embankment dams was provided by Sherard in his book “Embankment Dams (1963)”, and several figures adapted from that reference are shown in the following Figures 15, 16, 17, 18.

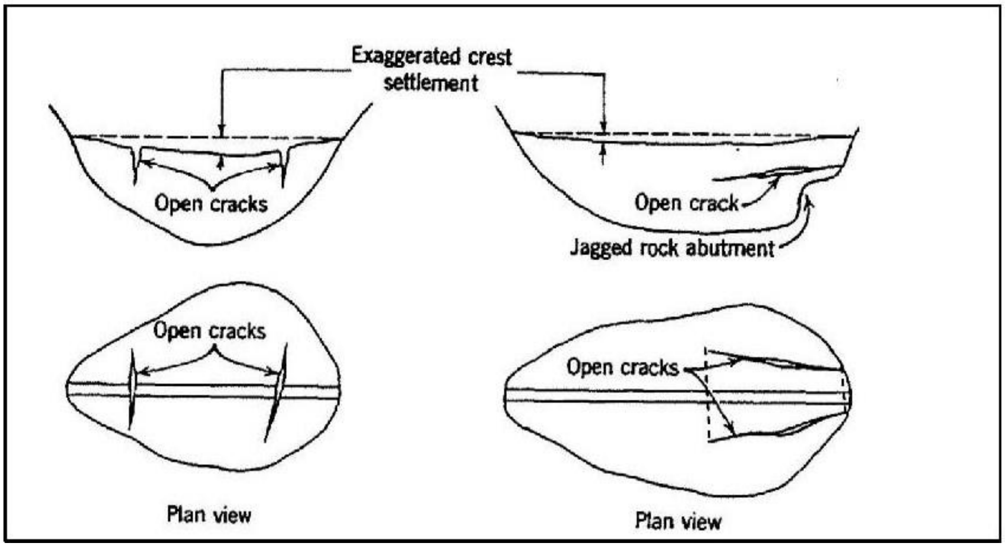


Figure 15: Transverse and longitudinal differential settlement cracks.

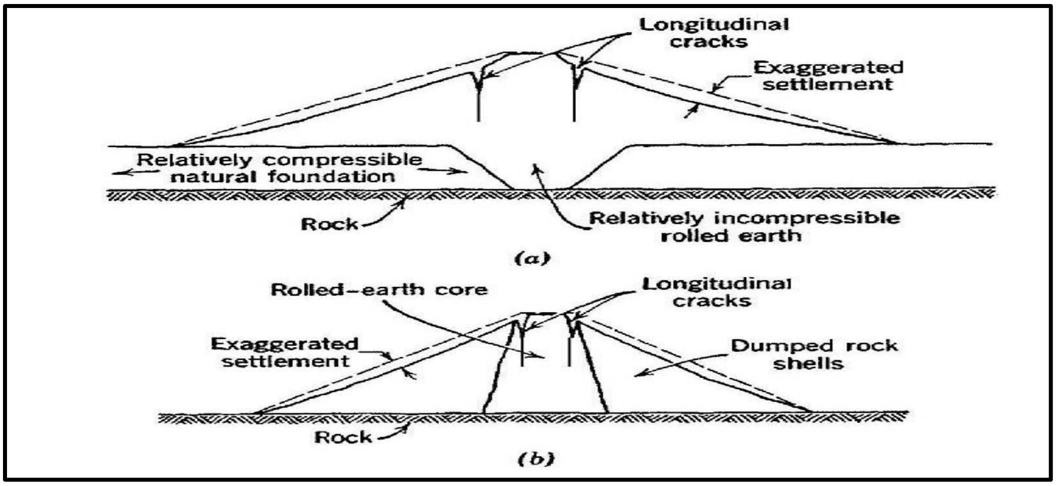


Figure 16: Longitudinal cracks (a) Cracking caused by differential Settlement. (b) Cracking caused by differential settlement between embankment sections of dumped rock and rolled earth.

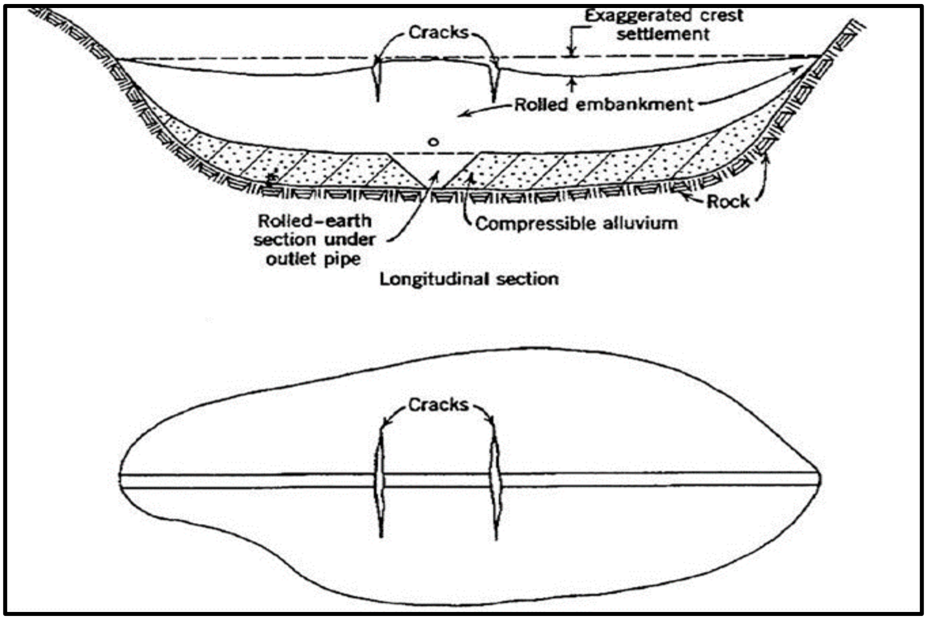


Figure 17: Cracking due to differential settlement between natural foundation soil and rolled-earth support under outlet pipe (or other discontinuity in the foundation).

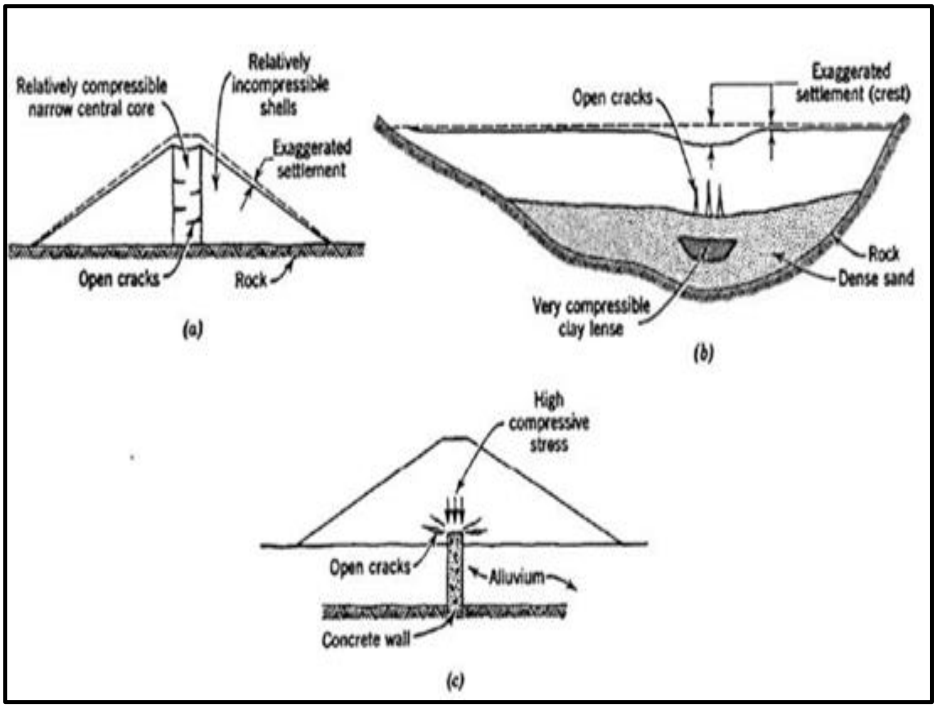

Relatively compressible narrow central core
Relatively incompressible shells
Exaggerated settlement
Open cracks
Rock
(a)
Open cracks
Exaggerated settlement (crest)
Rock
Dense sand
Very compressible clay lense
(b)
High compressive stress
Open cracks
Alluvium
Concrete wall
(c)

Figure 18: Internal embankment cracking.

Internal erosion in cracks due to differential settlement was not well understood by dam engineers in late 1800s. But, as it is clear now, this settlement may have been the prime cause leading to internal erosion and failure of one of the classic cases, which had occurred in England in 1864, i.e. Dale Dyke Dam failure. The Dale Dike Reservoir is a reservoir 13km away from the city of Sheffield, on a tributary of the River Loxley. It was in 1864; the Dale Dyke Reservoir's dam failed when the reservoir behind it was being filled for the first time. Around 244 people died, and 600 homes were destroyed. It had impounded 3,240 x 106m^3 when it failed; a crack in the embankment led to the breach, although this was not established at that time [17], [18].

Another source put the scores of failure at 238 people 700 animals, with 130 buildings destroyed and 500 partially damaged, in addition 15 bridges were swept away. It adds “The tragedy led to changes in civil engineering methods during reservoir projects. Several flaws in the dam’s construction and design were highlighted, and new safety standards were implemented. A new dam wall was built 600 meters further up the valley in 1875. A memorial stone marks the spot where the original dam stood” [19].

In Figure 19, the red line indicates the site originally chosen for the dam axis; however, it was soon discovered that this line lay in the middle of an old landslide area. The dam center line was consequently, moved about 150 yards upstream at the north end.

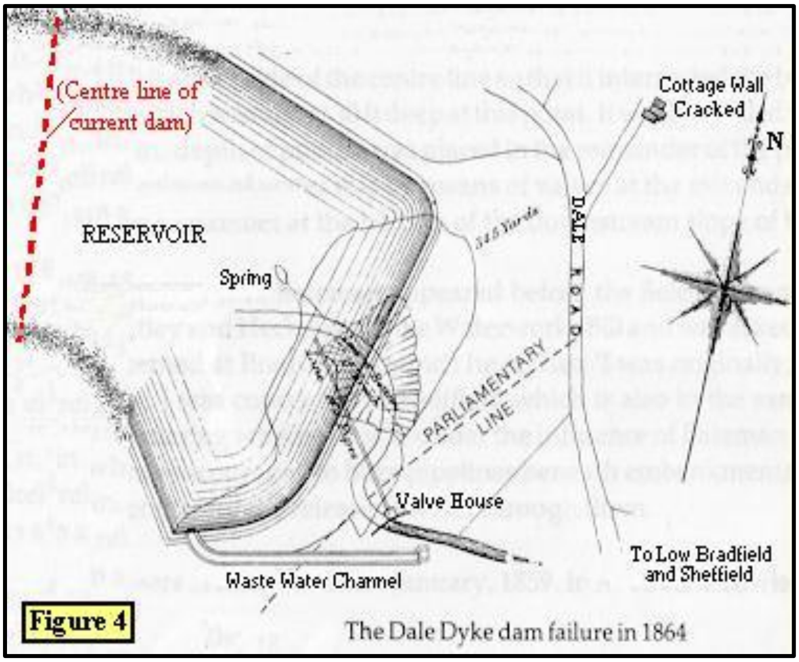


Figure 19: Dale Dike Dam’s layout.

A cross section of the original Dale Dyke Dam as designed by John Towlerton is shown in Figure 20. The dam had two outer shells with the outside slopes of a (2½:1) which were built from shale in the lower half with rubble stone and shale in the upper half. The central core was made of puddled clay core. The purpose of this puddle clay core was to provide a watertight membrane, essential to make the whole structure leak-free, and thus avoid any gradual erosion of the embankment. To ensure water-tightness, the foundation of this puddle clay core was built on bed-rock (“watertight strata”), and in the case of the Dale Dyke Dam, the engineers had to dig down to the unusually great depth of 60ft to reach solid rock, as indicated in Figure 20.

The puddle clay core of the dam stood 95ft above ground level, it was 16ft wide at the ground level, tapered to 4ft wide at the top, and as already mentioned, was sunk to a maximum depth of 60ft below ground, the crest being 12ft wide and 1254ft long. The dam had two 18" diameter outlet pipes, both sunk in trenches beneath the embankment, and terminating at a “Valve House” at the bottom of the downstream slope of the embankment.

When the unthinkable happened, 700 million gallons of water descended down the valley, sweeping through Loxley, Malin Bridge, Hillsborough, Attercliffe and even reaching as far as Rotherham. The flood passed the present-day sites of Don Valley Stadium, Sheffield Arena and Meadowhall. The “wall of water” destroyed everything in its path. The city center escaped damage, but the densely-populated Wicker area was wiped out.

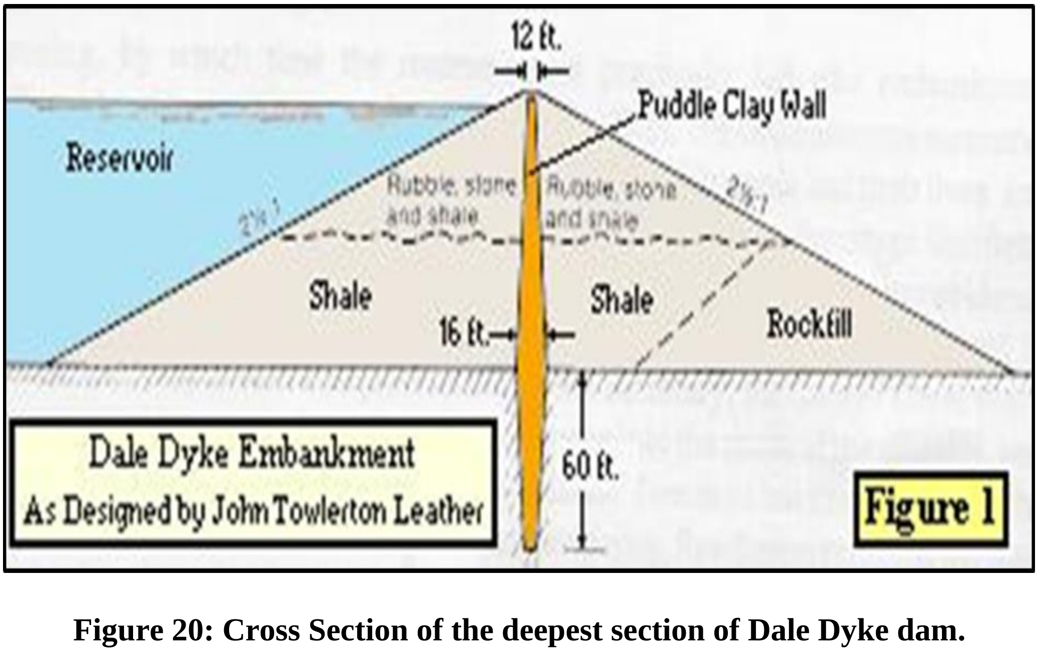


Figure 20: Cross Section of the deepest section of Dale Dyke dam.

Just after the failure, two investigation panels of leading dam engineers examined the site and listened to witnesses and submitted their reports to the authorities. One of the panels went further to uncover and inspect the pipes forming the outlets to the reservoir water, but the two panels came to different conclusions. The first attributed the failure to various aspects of “bad design and workmanship”, and it criticized the mode of laying the outlet pipes set in trenches beneath the embankment. The possibility of a fractured pipe, leading to leakage and consequential erosion of the embankment was taken as the reason for the disaster. The pipes, however, remained buried under tons of embankment: they could have been damaged in the collapse, in which event it would be difficult to prove the point one way or the other. The second panel stated that the catastrophe was most probably caused by a landslide; basing this on the fact that the downstream toe of the dam was built on an old landslide area, suggesting that the old landslip had moved again.

The inquest jury had little option but only to find that “there has not been that engineering skill and that attention to the construction of the works, which their magnitude and importance demanded”; adding in the final statement that “no one should be prosecuted”, so the case was closed at that.

It was not until an extensive study made by, G. M. Binnie in late 1970s of all the recorded evidence concerning the design, construction and destruction of the Dale Dyke Dam that the real cause of failure was revealed. He submitted a written report at a meeting of the Engineering Group of the Geological Society, London, on the 10th January 1978 in which he concluded that the dam's puddle clay core (watertight membrane in the center of the dam) had ruptured resulting in leakage and the consequential erosion of the inner central part of the embankment. Two of the dam’s original drawing which were discovered the following year gave all the answers to the cause of core rupturing; these are shown in Figure 21a and Figure 21b. A report upon the cause of the rupturing of the puddle clay core was then presented in Binnie’s book of that year; “Early Victorian Water Engineers”.

In his search for the original drawings of Dale Dyke Dam, Mr. G. M. Binnie came across the drawing shown in Figure 21a, which is of a longitudinal section on the axis of the original Dale Dyke Dam. It can readily be seen from Figure 21a and Figure 21b, that close to the center of the bottom of the puddle trench (the bed-rock on which the puddle wall was built), there is a very unusual vertical step in the rock formation of 35 feet depth (marked X in the drawing).

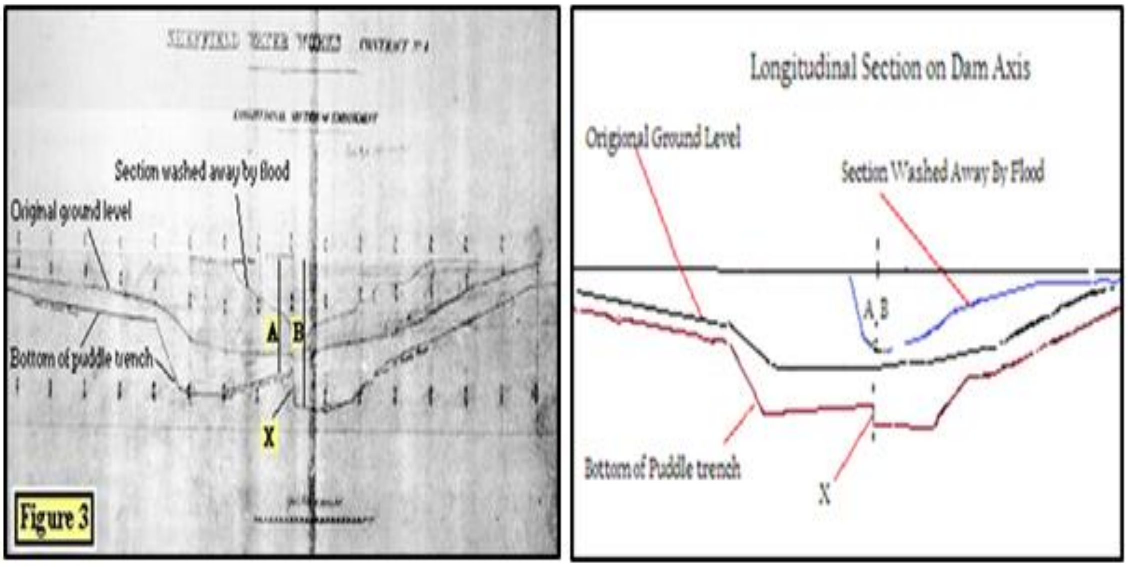


Figure 21: (a) The original drawing showing longitudinal section of the dam along its axis; (b) An illustrating sketch of (a).

It is also seen that this lies directly beneath the central part of the embankment which was washed away in the flood. It was this that led Binnie to declare: “we need look no further for the basic cause of the accident”: He went on to explain that this abrupt change in the puddle trench “must have caused a rupture in the puddle clay wall, most probably before the core reached its full height”, and that the differential settlement between the greater depth of the puddle wall at (B) compared to that at (A) would have resulted in the clay wall cracking just above this vertical step (marked by X in the drawing). No details of this irregularity in the rock foundation were revealed at the inquest nor in subsequent discussions.

All the evidence points to leakage having taken place due to the puddle wall being ruptured near its base, and this having occurred long before the dam's completion. Having clearly determined this, the postulated sequence of events that would have subsequently occurred leading to the eventual collapse of the dam can now be explained and as illustrated in Figure 22:

Stage 1: A cavity forms in the upstream side of the embankment due to erosion caused by leakage through the cracked puddle core.

Stage 2: The roof of this cavity progressively caves in resulting in the cavity migrating upwards. As a consequence of the reduced support against the clay core (possibly combined with the undermining of the core by the crack itself), the upper part of the crest leans in the upstream direction (towards the water); thus 'stretching' the outer slope of the embankment, and resulting in a horizontal crack forming a little way down the slope. This was the crack that was discovered earlier on the day that the dam collapsed.

Stage 3: When the migrating cavity reached the surface a “swallow hole” was formed very close to the crest with the consequence that the adjacent material slid into it. This is the stage when the waves suddenly started coming over the crest and down the crack [20].

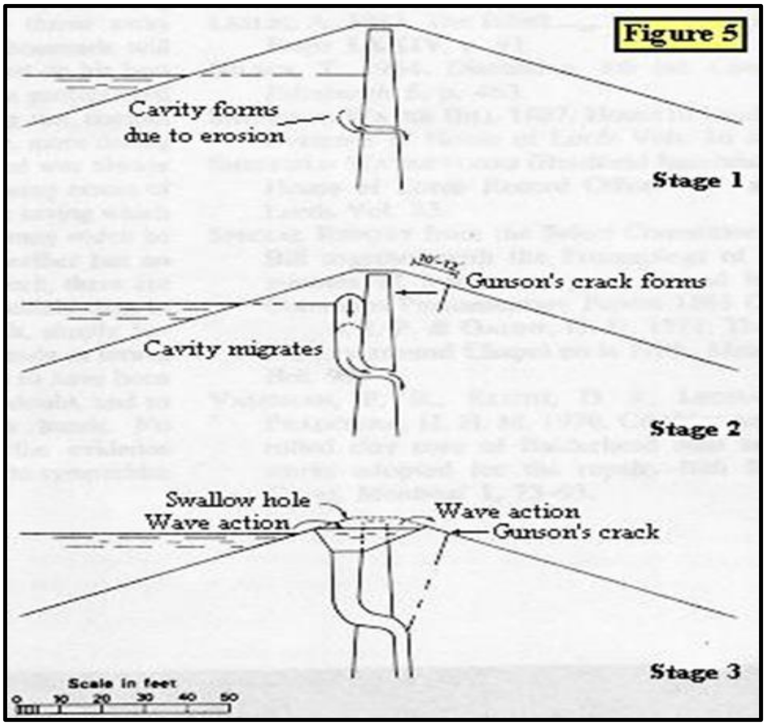


Figure 22: Stages of failure of Dale Dike Dam.

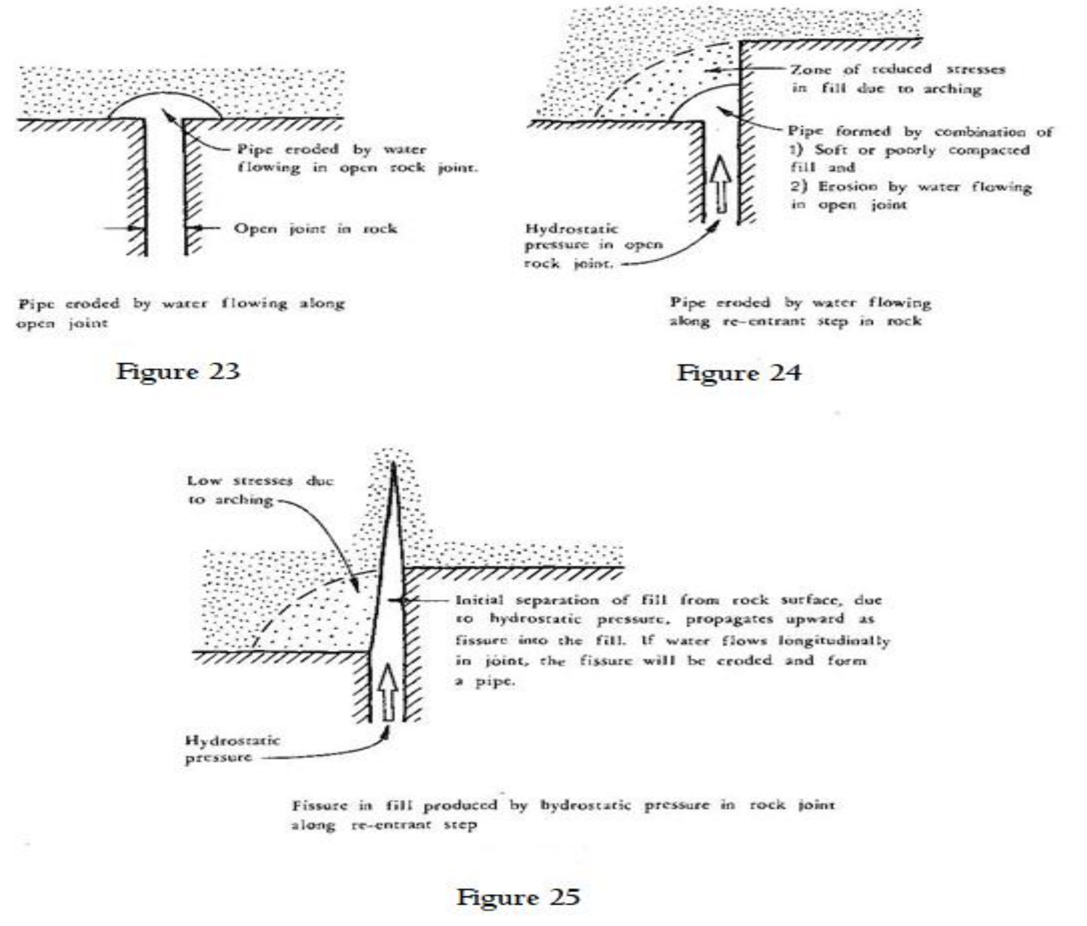
5.2.2 Hydraulic Fracture

Hydraulic fracturing is one of the important potential causes for leakage of earth dams, and it may be defined as “the condition leading to the creation and propagation of a thin physical separation in a soil or rock mass as a result of fluid pressures, which are excessive in comparison to the stresses within the soil or rock mass”.

Hydraulic fracturing in dams is likely to exist or develop along structures such as conduits, walls, vertical rock faces in foundations, and other locations where stresses could be low. This condition was identified in some failure cases where an outlet structure had been constructed in a trench excavated in the foundation, and the narrow space left between the structure and sides of the trench had made compaction of the fill around the structure difficult. In such a case, low-density, low-strength fill more susceptible to hydraulic fracture was more likely to result. Unfavorable geometry may result also in poor compaction that can prevent the full stress from overlying fill from being transferred to a conduit or other structure backfill. Similarly, arching in soil fill may transfer stress to the sides of the trench and not to the fill around it, or even to fill placed in or adjacent to sharp changes in foundation profile. In these situations, it is possible for the hydrostatic pressure to exceed the lateral stress in the backfill. The same phenomenon can occur next to overhanging or near vertical rock excavation surfaces or embedded structures. Hydraulic fracturing has been suggested to possibly have played a role in the failure of Teton Dam (USA 1976), which featured a very narrow, steep-sided cutoff trench backfilled with low plasticity soil. Studies carried out by the Independent Panel appointed to investigate the failure suggested this possible scenario for inducing internal erosion by the seepage water at the contact surface of the cutoff trench within Zone (1) of the core material at (St 14+00). Hydraulic fracturing might have occurred across the grouting cap and penetrating the core material. The performed studies were supported by hydraulic fracturing tests performed in boreholes at the Panel request in sections of the remaining fill with similar geometry of key trench and overlying fill to that at (St. 14+00). The field tests result at (St. 26+00) which was similar to the fill in (St. 14+00) indicated that the hydraulic fracturing was responsible for initiating internal erosion. According to this evidence, the Panel came up with the following postulations:

1. Fill material in Zone (1) of the core material at (St.14+00) was highly erodible. In such case, if the water pressure exceeds the intergranular pressure, tension develops in the soil skeleton, and if the tension exceeds the tensile strength of the soil, the soil may crack by the process known as hydraulic fracturing. Such condition could have occurred near the base of the key trench near (St.14+00) which was responsible for the original breach of the key-trench fill.
2. The mechanism of erosion under these conditions is illustrated by Figure 23, which shows an idealized joint in the bottom of the key trench. The joint is not sealed by dental concrete or slush grout; consequently, horizontally flowing seepage water under pressure would attack the base of the fill and begin to form a pipe. If the joint occurred at a step in the rock surface, Figure 24, the erosion would occur even more readily because of the reduction of stresses in the re-entrant corner due to arching, and because of the likelihood of poor compaction of the fill in the corner. Furthermore, under high water pressure, the pipe is likely to enlarge by separation of the fill from the rock surface, as illustrated in Figure 23, Figure 24 and Figure 25.
3. Conditions corresponding to the above mentioned mechanism have developed in the key trench near (St. 14+00) as inferred from hydraulic fracturing tests performed in (St. 26+00) already mentioned.
4. Seepage flow as demonstrated by field investigations clearly indicated that openings or windows existed in the grout curtain near the failure section, particularly at shallow depth beneath the grout cap. Even modest seepage beneath the grout cap, can develop into larger cavities upstream and downstream of the grout cap. These cavities may unite under the high hydraulic gradient between them to form a single erosion tunnel, Figures 26, 27 and 28. After this occurs, enlargement of the tunnel is restricted only by the capacity of the adjacent joints to deliver and carry away more of the

through-flowing water. The Panel's investigations leave no doubt that all the conditions, including hydraulic fracturing, leading to the creation of the initial breach by internal erosion were present [21].


Pipe eroded by water flowing in open rock joint.
Open joint in rock
Zone of reduced stresses in fill due to arching
Pipe formed by combination of 1) Soft or poorly compacted fill and 2) Erosion by water flowing in open joint
Hydrostatic pressure in open rock joint.
Low stresses due to arching
Initial separation of fill from rock surface, due to hydrostatic pressure, propagates upward as fissure into the fill. If water flows longitudinally in joint, the fissure will be eroded and form a pipe.
Hydrostatic pressure

Pipe eroded by water flowing along open joint

Figure 23

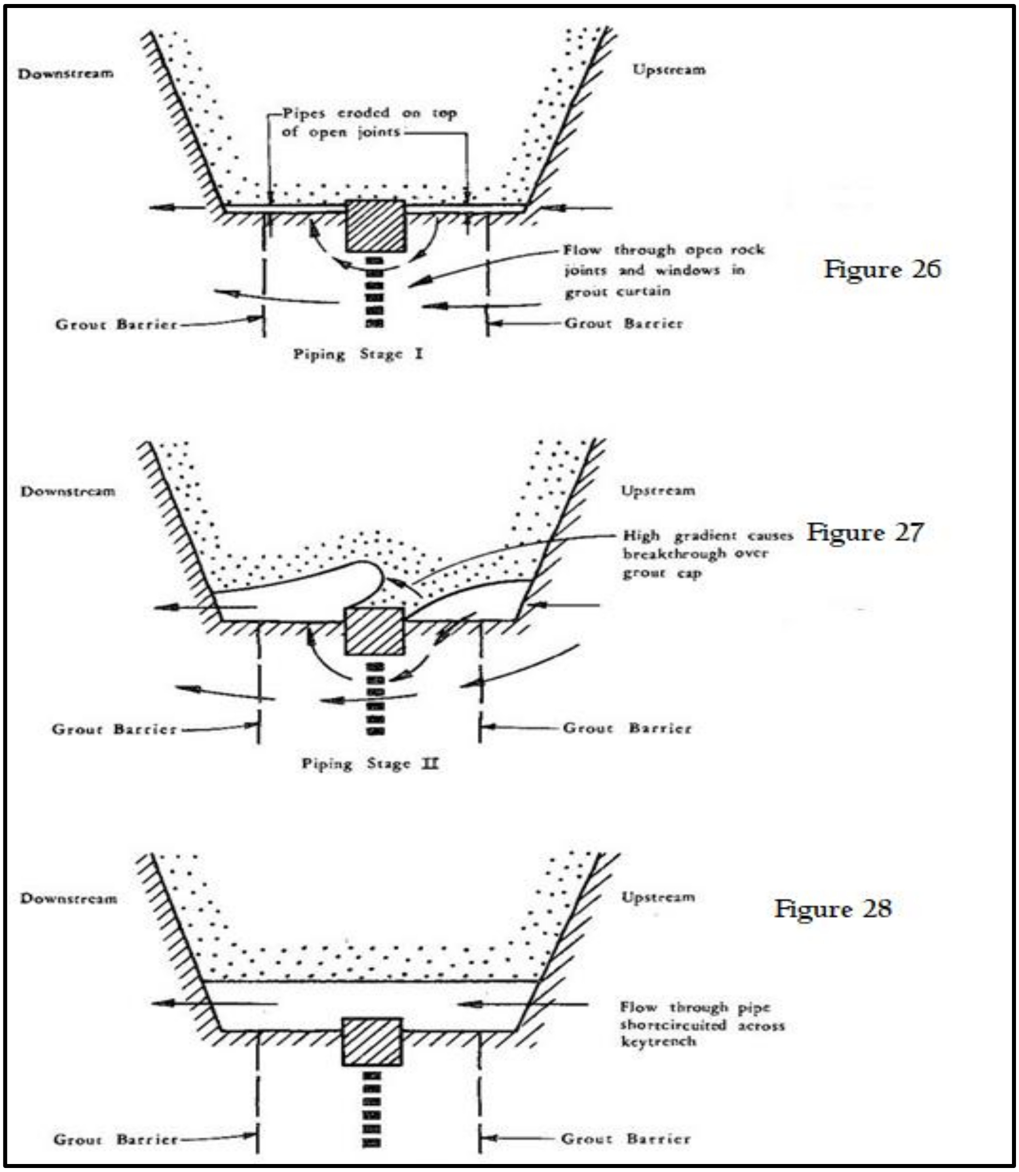
Pipe eroded by water flowing along re-entrant step in rock

Figure 24

Fissure in fill produced by hydrostatic pressure in rock joint along re-entrant step

Figure 25

Figures 23, 24 and 25: Development of internal erosion by arching leading to hydraulic fracturing [21].


Downstream
Upstream
Pipes eroded on top of open joints
Flow through open rock joints and windows in grout curtain
Figure 26
Grout Barrier
Grout Barrier
Piping Stage I
Downstream
Upstream
High gradient causes breakthrough over grout cap
Figure 27
Grout Barrier
Grout Barrier
Piping Stage II
Downstream
Upstream
Figure 28
Flow through pipe shortcircuited across keytrench
Grout Barrier
Grout Barrier

Figures 26, 27 and 28: Seepage flow beneath the grout cap developing into larger cavities upstream and downstream of the grout cap to form single erosion tunnel. Hydraulic fracture of soil around grout cap encourages the formation of erosion tunnel as in stage III.

5.2.3 Dam Filling Materials

Homogenous type of dams is built normally from low permeability materials to reduce seepage as much as possible in addition to having good strength parameters, which make them able to develop a maximum practical shear strength under compression and maintain most of it after the filling of the reservoir. They are designed to maintain fairly stable conditions under all operation modes. The use of such dams, however, is limited to small dams; therefore, the need to build larger and higher dams led to the development of zoned earth fill dams. In homogenous dams the seepage phreatic surface daylights in the downstream face at a height of about one third of the hydraulic head on the dam; refer back to Figure 5. If toe drainage is used such as in Figure 6, then the phreatic surface is lowered, and a safer condition of stability is obtained. Another arrangement for lowering the phreatic surface may still be obtained by installing a drainage blanket as in Figure 7, and when the chimney drain is added to the drainage blanket then this arrangement looks like Figure 8.

In zoned earth fill dams, the central zone is normally built of low permeability core to reduce seepage through the dam to the minimum possible level. Such cores are usually supported by upstream and downstream outside shells of more pervious material to give weight and stability to the dam under various operation conditions. The shells may vary in composition according to each case from sand-gravel fill, to rockfill, or random fill. Moreover, transition zones of filter materials are included to separate between the core on one side, and the upstream and downstream shells on the other to control seepage and leakage passing through these shells and prevent sediment transport through any cracks in the central impervious core. Additionally, these filters tend to stop any newly formed cracks to dangerous dimensions by filling them in a self-healing process, if such cracks are formed during the life of the dam due to any of the reasons explained previously. Various arrangements to discharge safely the seepage water from the filter zones may be constructed, such that the downstream filter zone is made in the form of chimney drain connected to a drainage layer at the dam base or by adding a toe drain of sufficient height at the toe to collect seepage water directly. Figure 29 and Figure 30 shows such arrangement in a zoned earth fill dam [22].

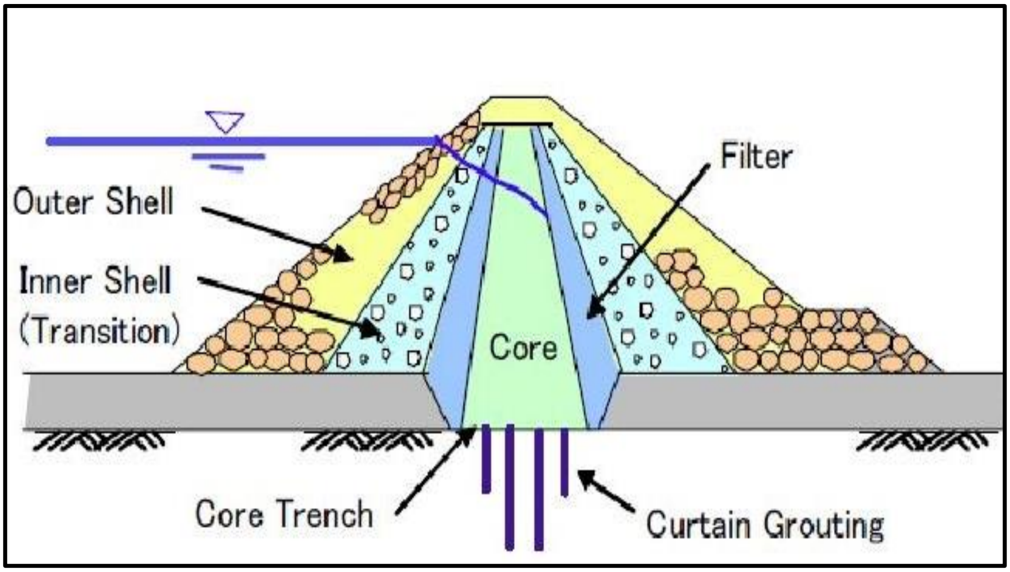


Figure 29: Rockfill Dam with clay core and filter zones.

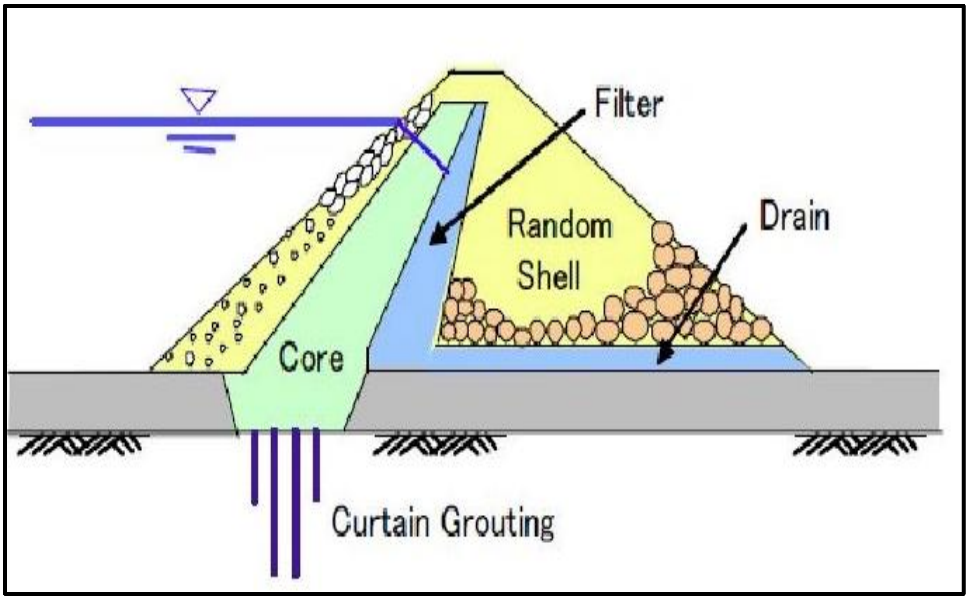


Figure 30: Rockfill Dam with Chimney filter and drainage layer.

Filters and drainage layers' design are ruled by strict guidelines, which have been developed over the years and experience gained since the use of these materials was proposed by Terzaghi, but the selection of clay core materials and the shell materials are governed by their availability at the site.

With the experience gained so far in embankment dams, the design philosophy has changed from minimizing seepage to controlling seepage and lowering the saturation of the phreatic surface by incorporating filters and drainage elements thus permitting steeper slopes. This kind of changes, including drainage features makes it possible to build a dam with thinner impermeable cores at places where soils of low permeability are available in small quantities, or even using soil of inferior quality but using concrete or asphaltic concrete diaphragms with them leading to considerably greater cost.

5.2.4 Clay Materials

In some cases, the choice of low permeable soil to build homogenous embankments or impermeable core for zoned dams may play an important role in the development of the internal erosion process under seepage conditions. One example of such material is dispersive clay, which should be avoided or properly treated. The process of dispersion occurs where clay particles are able to move about freely as colloidal solution because they are not bound to other clay particles nor to organic matter, which makes them tunnel prone and can raise problems for many earth structures. In appearance, they are like normal clays that are stable and somewhat resistant to erosion, but in reality, they can be highly erosive and may cause severe damage or even failure.

Dispersive clays differ from ordinary erosion resistant clays since they have a higher relative content of dissolved sodium in the pore water. Ordinary clays have a preponderance of calcium, and magnesium dissolved in the pore water. Normal clays have a flocculated or aggregated structure because of the electrochemical attraction of the particles to each other and to water. This accounts for these soils' cohesive, nonerosive behavior. Dispersive clays have an imbalance in the electrochemical forces between particles. This imbalance causes the minute soil particles in a dispersive clay to be repulsed rather than attracted to one another. Consequently, dispersive clay particles tend to react as single grained particles and not as an aggregated mass of particles. Dispersive clays are most easily eroded by water that is low in ion concentration, such as rain water or reservoir water. Typically, dispersive clays are low to medium plasticity and classify as CL in the Unified Soil Classification System (USCS). Other USCS classes that may contain dispersive clays are ML, CL-ML, but CH. Soils classifying as MH rarely contain dispersive clay fines [23]. Because of many failures of earth dams and their serious consequences which have resulted from internal erosion and piping, empirical criteria were developed before the mechanism of piping was understood. Many laboratory studies of piping in cohesive soils were initiated to determine the mechanism of such failure. Studies revealed the unique characteristic of dispersive clays as a particular type of soil in which the clay fraction erodes in the presence of water by a process of deflocculation. This occurs, as explained, when the interparticle forces of repulsion exceed those of attraction so that clay particles are detached and go into a colloidal suspension. If the water is flowing, as in a crack within an earth dam, the detached clay particles are carried away, and internal erosion develops.

Research on failures in earth dams, due to dispersive clay behavior, was initiated in Australia in 1960. The first study of dispersive clay piping in earth dams in the

United States was reported in 1972, which developed a relationship between percent sodium and total soluble salts in the soil pore water extract and field performance of earth dams as evidenced by piping failure or rainfall erosion damage. One procedure for identifying dispersive clays, known as the Pinhole erosion test, was devised by the Waterways Experiment Station (WES). A series of laboratory tests has been designed to standardize a procedure for the use of the pinhole erosion test as a method of identifying dispersive clays, evaluate the effectiveness of filters in preventing piping in dispersive soils, and determine the influence of selected parameters on erodibility of dispersive clays [24].

The tests most commonly used for identifying dispersive clays are the crumb test (ASTM D 6572), the double hydrometer test (ASTM D 4221), and the pinhole test (ASTM D 4647) where the pinhole test is a direct measure of the erodibility of soil. Figure 31 shows an arrangement for Pinhole test apparatus.

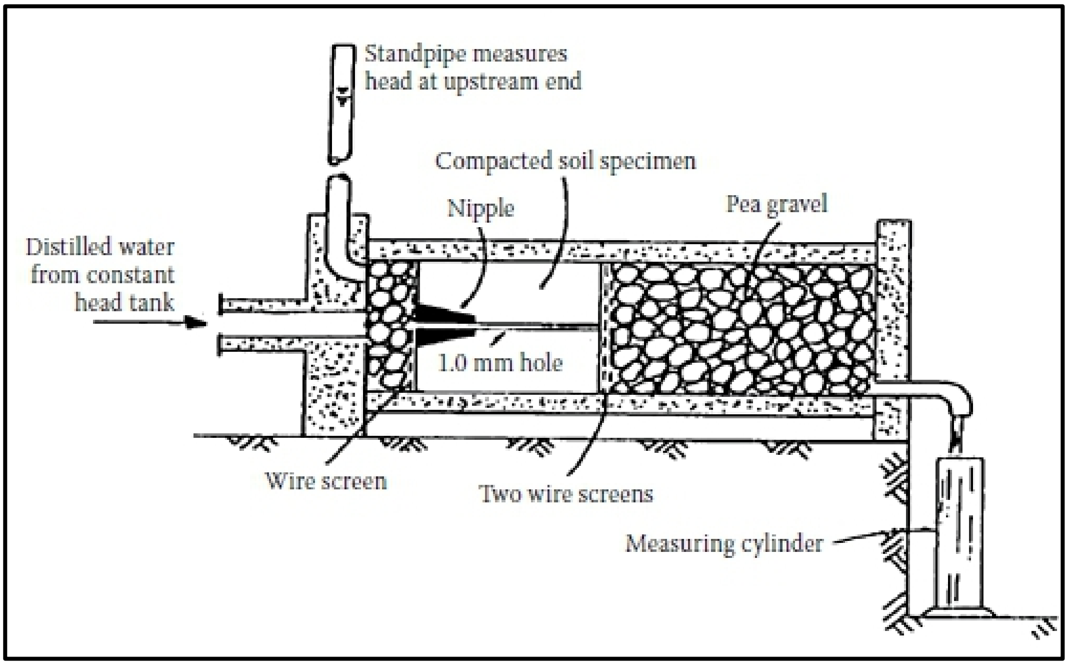


Figure 31: Pinhole test apparatus for dispersive clays.

More information on all these tests may be drawn from report number R-91-09 of the Unites States Bureau of Reclamation, titled “Characteristics and Problems of Dispersive Clay Soils” [25].

In embankment dams, if the low permeability core, which is normally of clayey soil develops cracks, particularly transverse cracks, from hydraulic fracture, desiccation, or other causes as described already in the previous paragraphs, the integrity of the embankment dam may be at risk. Water flowing through a crack in any soil will erode and enlarge the crack, unless the crack is able to swell closed before erosion occurs. If the crack continues to erode, this can lead to a breaching of the embankment dam. Figure 32 and Figure 33 illustrates failures known to be associates with highly dispersive clay embankments. Both figures show a small embankment dam that failed when water flowed along a transverse crack in the dam. The transverse crack was caused by hydraulic fracture of the earth fill associated with differential settlement near the conduit. Failures of embankment dams constructed of dispersive clays without appropriate defensive design measures have been common, refer to Figure 32.

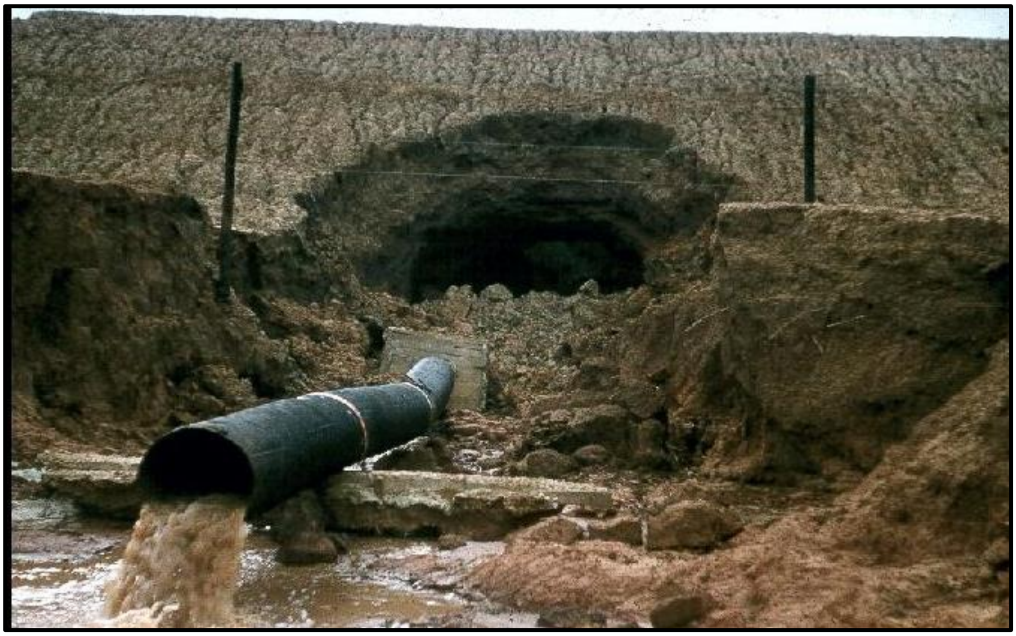


Figure 32: An embankment dam in Mississippi constructed of dispersive clay soil, failed on first filling with low reservoir level. Failure was likely caused by cracks from differential settlement, hydraulic fracture, or poor compaction about the outlet works conduit. The embankment dam had no filter and the clay was dispersive.

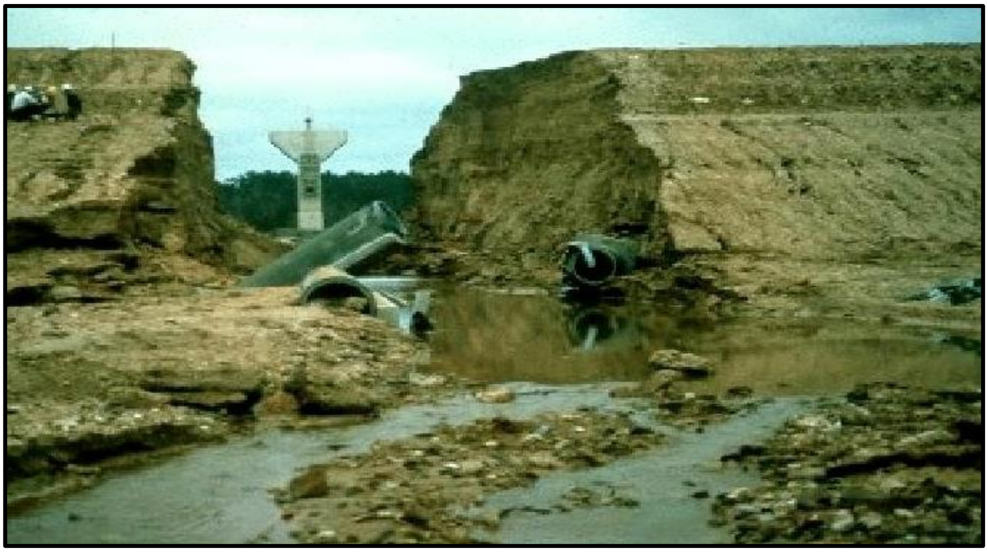


Figure 33: The embankment dam shown was constructed with dispersive clays, failed on first filling of the dam.

Concluding, it may be said that, the erosion resistance of impermeable cores depends on several factors, including the gradation, degree of compaction and compaction water content, plasticity and electrochemical composition. The most erosion-resistant zones are high in plasticity with an electrochemical composition that results in strong interparticle attraction, compacted to a high percent saturation to reduce their permeability. The least erosion-resistant soils are “dispersive soils” with low plasticity indices, which are also more likely to experience rapid internal erosion than soils with higher plasticity. If the use of dispersive clay in dam cores is unavoidable, then ample of properly designed filter zones may be used to guard against internal erosion that may develop in such cores. An embankment chimney drain/filter that extends completely across an earth fill dam, from one abutment to the other and extending upwards to the normal pool level or higher, is often used. Similarly, filter around conduits is important where dispersive clays are used for embankment dam construction. The dimensions of the filter collar should be increased since dispersive clays are so dangerous to the integrity of an earth fill, especially if a seepage path may develop along the contact surface with the structure leading to internal erosion [26].

5.2.5 Gap Graded Soils Material

Another process associated with seepage action is known as suffusion. This mode of internal erosion may be associated with a gap-graded sand-silt or sand-silt-gravel fills that are used in construction of the outer shells of dams, or they could be present in foundations of such dams. Care must be exercised, also, to avoid them in filters. Suffusion occurs when fine particles of a soil mass of the types mentioned above, are moved through voids between larger soil particles by seepage forces within the soil mass. Suffusion can only occur provided the fine soil particles are small enough to pass between the coarse particles and do not fill the voids in the coarser soil. Soils susceptible to suffusion are described as internally unstable. Water flow velocity must also be sufficient to transport those fine particles. Suffusion occurring within an embankment shell or the foundation of a dam will result, after the fine fraction is removed, in a coarser soil structure leading to increased permeability and seepage, settlement of the embankment in the dam foundation, and possibly hydraulic fractures and a higher likelihood of slopes instability, which can lead to failure of the dam.

Much research based on laboratory testing was conducted during the 80s and 90s of the last century. This research on sand-gravel soils indicates that for suffusion to occur, the following three conditions have to be satisfied:

1. The size of the fine soil particles must be smaller than the size of the constrictions between the coarser particles, which form the basic skeleton of the soil.
2. The amount of fine soil particles must be less than enough to fill the voids of the basic skeleton formed by the coarser particles. If there are more than enough fine soil particles for void filling, the coarser particles will be “floating” in the matrix of fine soil particles, instead of forming the basic soil skeleton.
3. The velocity of flow through the soil matrix must be high enough to move the loose fine soil particles through the constrictions between the larger soil particles [27].

Some embankment dams contain zones of broadly graded soil with sufficient fines content to be considered of low permeability as a whole, but the finer fraction of these soils are found to be subject to suffusion from internal instability. The internal instability of these soils results from the ability of finer soil particles to be mobilized between larger particles in these broadly graded soils as described already. Suffusion, if it is allowed to continue, can result in the formation of sinkholes if erosion progresses long enough. Using a graded filter in sufficient quantities and designed using current modern criteria has been shown to be effective in blocking internal erosion in these soil types.

Analysis of the suffusion phenomenon, which uses the particle size distribution to assess the internal stability of a soil, can directly predict the likelihood of occurrence or otherwise. There are now many approaches for doing so, including Kenney-Lau approach, which is a renowned method for predicting suffusion using particle size distribution to assess the likelihood of such occurrence [28].

5.2.6 Backward Erosion

Another form of internal erosion is the backward erosion piping process, which may start at an exit point in the downstream of earth fill structure and progresses backward (upstream) towards the reservoir. In this process, pipes or tunnels are formed in or underneath an earth fill dam or dyke. As seepage water flows through these pipes from the reservoir, it dislodges and washes soil particles from this fill depositing the eroded material at the downstream side of the structure. Figure 34 shows the various modes of backward erosion; note that the yellow arrows in this figure indicate the direction of seepage flow while the backward erosion process occurs in the reversed direction.

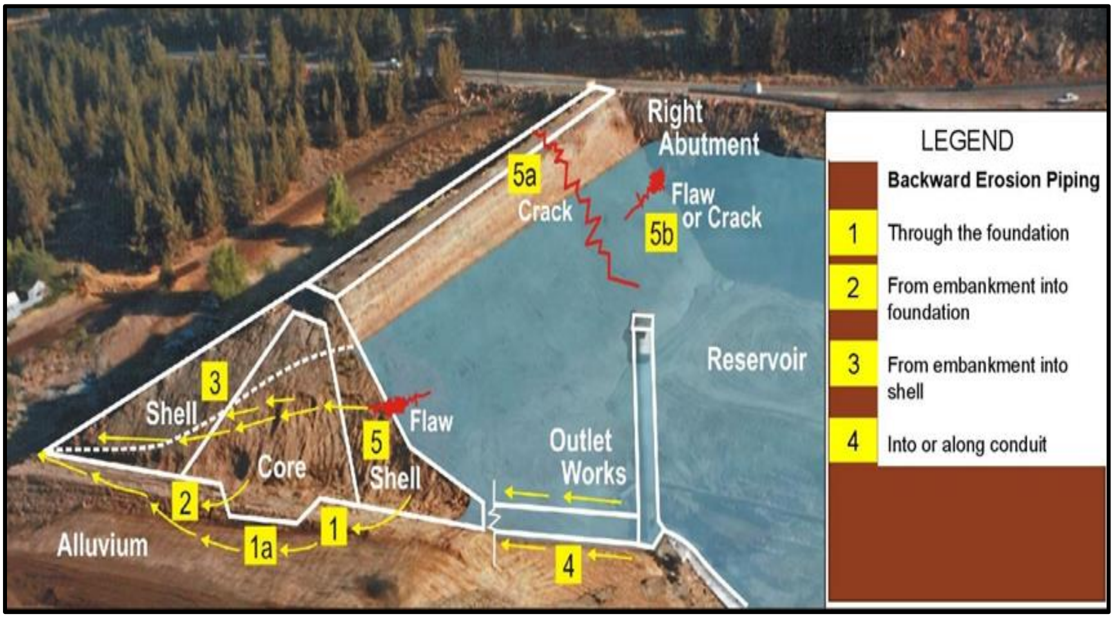


Figure 34: Modes of backward Erosion Piping [29].

For backward erosion to occur three conditions have to be satisfied:

1. Sufficient hydraulic head is necessary as the driving force.
2. Continuous layer susceptible to piping, beneath a layer or structure capable of forming a roof must exist.
3. An exit point, unprotected by filter is required for this type of erosion to start. If geologic or dam conditions are such that, a defect exists or likely to develop that increases pore pressures within the dam or foundation, then gradients at the seepage exit could increase sufficiently to initiate backward erosion piping where none had been observed previously.

Embankment dams and other structures founded on cohesionless soil deposits and not provided with a positive cutoff to bedrock, through which significant head loss is achieved, are most susceptible to piping through the foundation, unless they are provided with efficient drainage arrangement at the downstream.

High embankments constructed from cohesionless and /or low-plasticity cores can also be at risk for piping when they are not provided with filter zones within the fill. An illustration of backward erosion piping through foundation (Cases 1 and 2 shown in Figure 34) is illustrated in Figure 35.

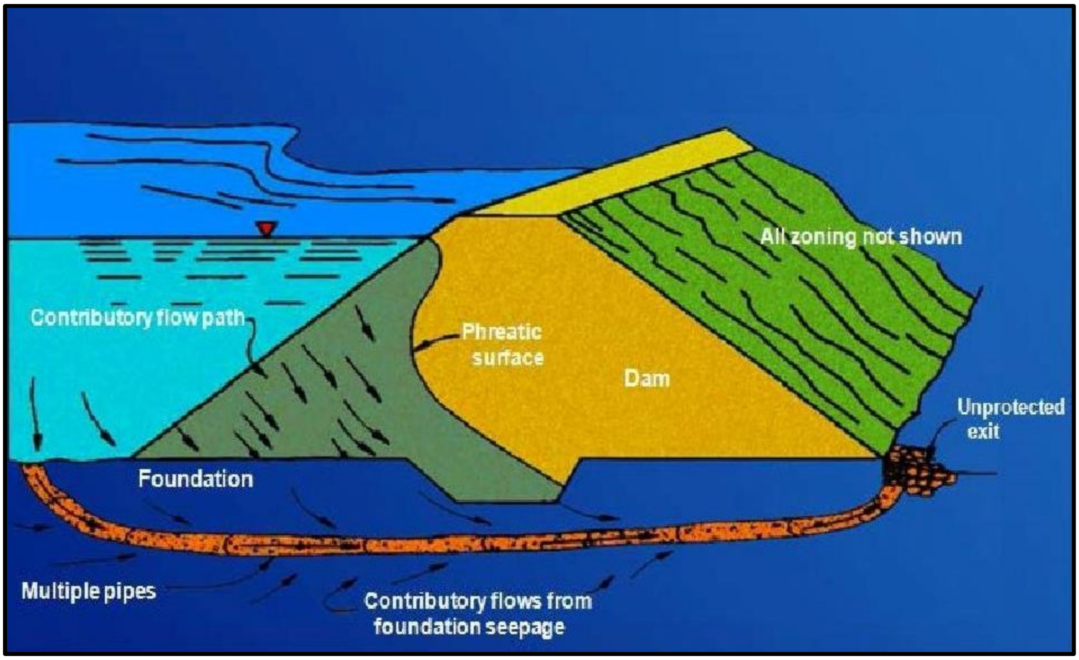


Figure 35: Backward erosion piping showing the fully developed pipe connected from downstream to upstream.

If filter zones are not provided within the earth fill, or if the downstream toe is not protected by drainage material, then there is the possibility of the phreatic surface daylighting on the downstream slope of the embankment providing conditions for the initiation of backward piping. Similarly, if ground water at the downstream rises to the ground surface, as the case may be if anti-seepage measures under the dam (cutoffs) are not provided or are inadequate, then the same conditions for backward piping exists. The critical exit gradient in such a case depends on the geology of foundation conditions and type of soil material at the exit i.e. particle size and plasticity of the soil, in addition to the hydraulic head at the exit point.

Backward erosion can also develop along conduits of outlet works crossing the foundation of an earth fill dam (Failure mode 4 in Figure 34). Factors that increase the likelihood of internal erosion and backward erosion piping incidents include:

1. Conduits constructed across abruptly changing foundation conditions, which are more likely to experience differential settlement such as concrete diaphragm or bedrock with a quickly changing profile).
2. Circular conduits constructed without concrete bedding or cradles are more likely to experience problems than conduits in more favorable shapes.
3. Conduits with an excessive number of joints are more likely to develop defects that can lead to problems.
4. Excavations made to replace unsuitable foundation materials for conduits increase the potential for differential settlement problems.

5. Conduits with compressible foundations are more likely to deform excessively, which may damage the conduit. Compressible foundations may also contribute to differential settlement that can result in hydraulic fracture of the earthfill surrounding the conduit.
6. Conduits located in closure sections in embankment dams contribute to differential settlement problems.
7. Embankment dams constructed with materials susceptible to internal erosion or backward erosion piping.
8. Conduits constructed without adequate compaction around the conduit,
9. Embankment dams constructed without a chimney filter or conduits constructed without a filter collar or filter diaphragm.
10. Conduits constructed of materials susceptible to deterioration, such as corrugated metal pipes [30].

Studies conducted on concrete dams on rock foundations have confirmed that seepage exit gradient and; therefore, the possibility of internal erosion developing and formation of cavities in such foundations, is controlled by the rock mass discontinuities, that are several orders of magnitude more permeable than the intact rock. It was demonstrated that such possibilities are mainly influenced by:

1. Variability of the joint apertures.
2. Degree of interconnection of the joints in a joint network within the rock mass, and the filling materials inside the cavities [31].

6. Uplift Problems under dams

Seepage water through or below dams can, in addition to other negative effects, exert uplift pressure on the dam base which may cause heave or blowouts at the toe, or even sloughing of the slopes in earth fill dams. When a dam is founded on relatively low permeability layer of soil or rock, then seepage through one or more pervious layers below this one generates relatively high uplift pressure on the confining layer. In this artesian condition most of the seeping water will pass through the permeable layers resulting in large percentage of the hydraulic head to act directly up on the confining layer.

If the remaining head at the toe of the dam exceeds the overburden weight of the overlaying layer, and if the upward flow of seepage water is strong enough, then it will breach the confining layer causing what is known as a blowout, whereby sand particles and other finer particles are washed out and deposited around the springs in a conical ring, referred to as a sand boil. Two idealized cases of piping at the toe of an earth dam resulting from excess uplift pressure are shown in Figure 36 and Figure 37.

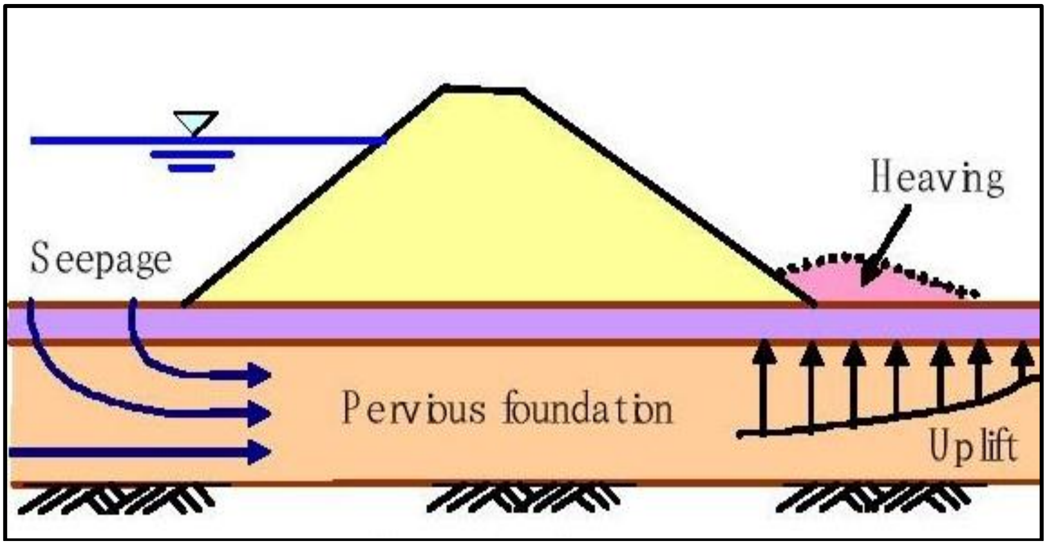


Figure 36: The case of dam founded on impermeable layer and permeable layer beneath it (artesian flow) [32].

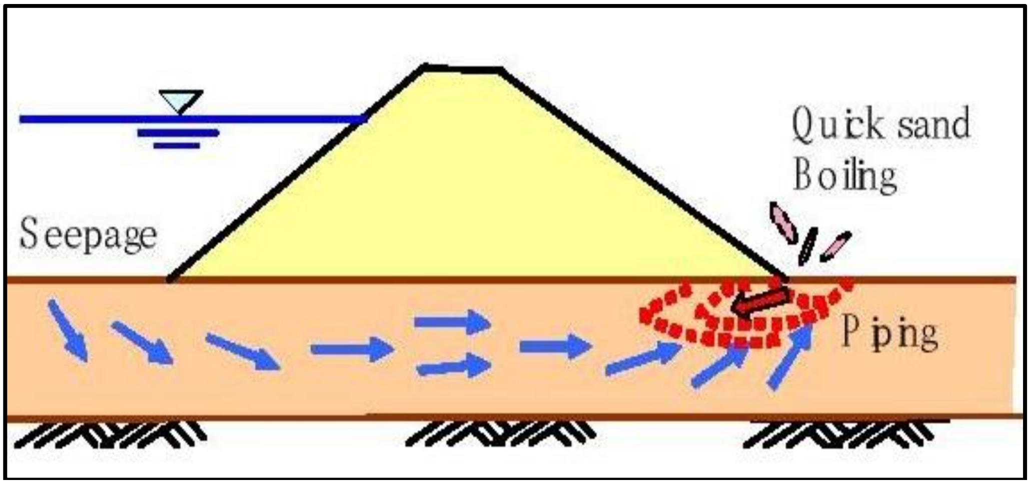


Figure 37: The case of a dam founded on permeable foundation (free surface flow) [32].

If a sand boil continuously removes material due to an excessive hydraulic gradient, they may eventually lead to piping, collapse or failure of the structure. Figure 38 illustrates various types of sand boils. Type A is indicative of a static condition for the current hydraulic gradient and is not necessarily indicative of an immediate problem developing. However, if the hydraulic gradient could increase during an extreme event, a Type A boil may become a Type B or Type C boil depending on the magnitude of the hydraulic gradient and soil conditions within the foundation or embankment. Type B is a boil that is carrying material, but the material is originating from near surface soils rather than deeper zones. A Type B boil may be indicative of a more serious problem developing that could warrant corrective action. Type C is indicative of a critical condition, where the present hydraulic gradient is removing subsurface soils. A Type C boil would need immediate action to remedy. Piezometers can be used to monitor downstream foundation uplift pressure and detect unsafe conditions before failure occurs. A key indicator of a potential problem developing is fine soil carried in water draining from a boil. In this case, the water is cloudy rather than clear or it may contain scattered fine particles [33].

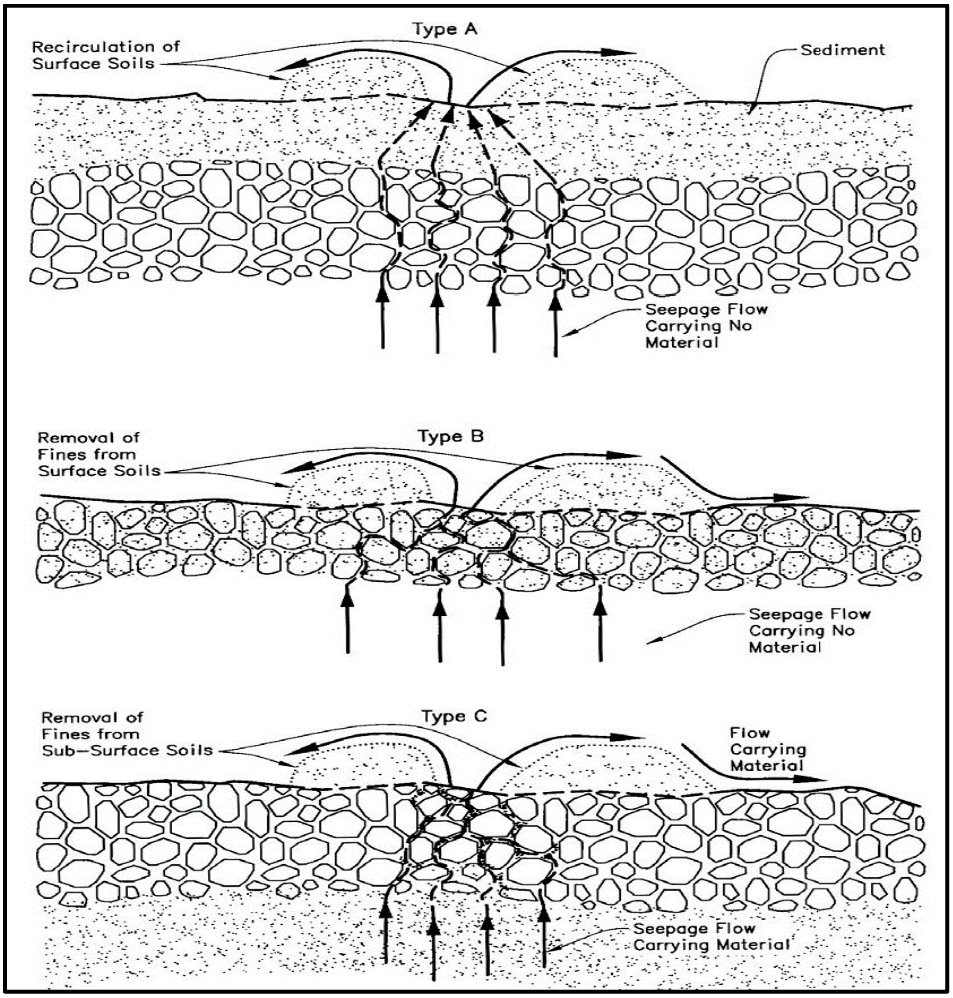

Type A
Recirculation of Surface Soils
Sediment
Seepage Flow Carrying No Material
Type B
Removal of Fines from Surface Soils
Seepage Flow Carrying No Material
Type C
Removal of Fines from Sub–Surface Soils
Flow Carrying Material
Seepage Flow Carrying Material

Figure 38: Types of sand boils likely to develop due to uplift at the toe of dam.

Generally, drainage facilities installed at the downstream of a dam are effective to safeguard against such conditions. Drainage methods usually can take the form of trenches, pervious blankets or berms, relief wells, drain holes and drainage tunnels or adits, which is applied to concrete gravity dams. Relief wells at the downstream toe of the dam are highly effective in relieving excessive uplift and potential piping forces. As compared to cutoffs, relief wells and most other drainage systems may marginally increase the loss of reservoir water and relief well discharge requires proper handling.

When confined flow conditions are predicted during design or develop during operation then one of the more effective safety precautions may be taken by installing relief wells at the toe of the dam spaced at calculated intervals; such an arrangement may be seen as in Figure 39.

In case of foundations, consisting of pervious material right from the surface to the impervious layer like rock (refer to Figure 37), relief wells may not be fully effective as the flow is being distributed over the entire length and depth of the foundation. Such relief wells, however, serve their purpose in relieving pressures that may get locked up below some impermeable pockets or where the horizontal permeability differs substantially from the vertical permeability. Hence, they should be generally provided in such cases, especially if the anti-seepage measures used for the dam (cutoffs, or blankets) only have been partially provided.

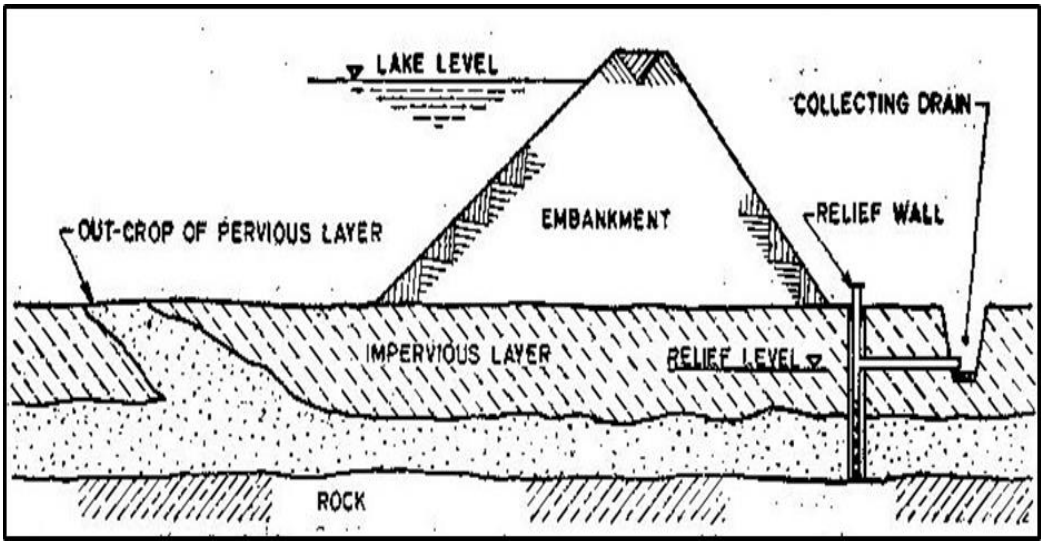


Figure 39: Typical Arrangement of Relief wells installed at the toe of earth fill dam.

Relief wells for dams may have specific problems associated with the plugging of the filter packs or the accumulation of bacteria or carbonates. Although relief wells are not typically installed for the primary purpose of seepage water monitoring, their outflow can be collected, measured, and evaluated similar to open piezometers or observation wells. Piezometers may be installed adjacent to relief wells to measure and monitor the pressure increases associated with clogged wells. Relief wells typically require regular long term maintenance for proper functioning, including re-development, chemical treatments, and disinfection [34].

The under layer(s) in many cases consists of fine-grained soils or sands, which may have often some degree of cohesion. In other cases, it may be of cohesive soils (clays). The behaviors of these types against the upward flow are different. In particular, clay particles are held together by cohesion and are not as easily dislodged from the soil mass as sand grains are from a layer of sand. Therefore, the ability of cohesive soil to resist uplift consists of two components: one due to its weight (similar to non-cohesive soils) and another due to its tensile strength. However, no widely accepted method for quantifying the contribution of tensile strength exists, and it is typically neglected in uplift analyses.

The full mathematical treatments for both confined and unconfined flow problems under both earth fill and concrete dams are presented in chapters 5 and 6 of the United States Army Corp of Engineers (USACE), EM 1110-2-1901, 30 September 1986 amended in April 1993, while treatment of flow towards wells is given in chapter 7 [34]. The reader is advised to refer to this publication for more elaboration on these mathematical solutions if needed.

In the case of earth fill and gravity dams founded on rock, it is customary to grout and drain the foundation rock of these dams. This practice works well for defective as well as sound formations and helps not only in reducing seepage quantities but enhance uplift pressure distribution under the dam and reduce it to safe limits and increase its factor of safety against sliding. In gravity dams, a well-planned, well-executed grouting and drainage program does not only reduce seepage through the rock, but also may disclose the presence of unsuspected weaknesses in the foundation, thus allowing for the improvement of any such existing defects.

The performance of large concrete gravity dams subject to uplift under seepage condition is controlled by the following:

1. Geological conditions of foundation rock.
2. Performance and effectiveness of grout curtains.
3. Drainage arrangement which works in conjunction with grouting.

These conclusions were highlighted in one report prepared by the Working Group on Uplift Pressures under Concrete Dams- ICOLD European Club, which is based on data from many dams' authorities.

The report concludes

First, that uplift pressures are controlled by the geology of the foundation in regard of rock mass discontinuities, which are several orders of magnitude more permeable than the intact rock. Moreover, uplift pressures are influenced by the variability of joint apertures and degree of interconnection of the joints in a joint network, and the different permeabilities along or across shear zones or faults.

Second, in old dams' construction practice, shallow concrete walls or cut-offs were often constructed near the heel of the dam to prevent high uplift pressure from being transmitted along any large open joint near the surface. In modern dams, grout curtains serve the same purpose. While it is agreed that a well-constructed grout curtain can reduce the amount of seepage through a dam foundation, the influence of the curtain on uplift pressures is still a topic of debate. This was clearly demonstrated by a study carried out by the Electric Power Research Institute (EPRI) and reported by the Working Group, which indicates that out of the 148 dams investigated, 70% of them had grout curtains, which had shown very variable situations. These ranged from excellent examples of grout curtain effectiveness to situations where the grout curtain had a negligible effect upon uplift. It was concluded, therefore, that grout curtains may be effective in reducing uplift but, in the absence of instrumentation to continuously prove that effectiveness; it is not prudent to rely upon the curtain for significant uplift reduction.

Finally, it was confirmed that for the study cases that were considered drainage was the single most effective mean of reducing uplift pressure, providing a direct highly permeable path between the water bearing discontinuities and the tailwater. In all the numerous cases examined, some measurable degree of drainage effectiveness was found, and in a good number of examples the installation of drains produced dramatic benefits [35]. In the United States, the three leading dam authorities, namely USBE, USACE and FEREC agree on the leading role that uplift plays in the stability of concrete dams, but they differ; however, in the way such uplift is accounted for in their analysis and on the effectiveness of drainage in general [36].

Seepage can also develop behind or beneath concrete structures such as chute spillways or headwalls. If the concrete structure does not have means such as weep holes or relief drains to relieve the water pressure, the concrete structure may heave, rotate, or crack. The effects of the freezing and thawing can amplify these problems. It should be noted that the water pressure behind or beneath structures may also be due to infiltration of surface water or spillway discharge but should still be addressed. One striking example is the recent failure of Oroville Dam, which threatened the failure of the whole dam leading to national catastrophe in California, USA [37] and [38].

7. Seepage Control Measures and Conclusions

Seepage control measures have been discussed already in the previous paragraphs, but a summary of these measures may serve to highlight the importance of such a measure in increasing dam’s safety against the actions of seepage, which are manifested in different forms according to type of dam, its foundation conditions and its material of construction. First of all, it must be said that seepage cannot be stopped altogether in all practical cases of dam construction.

The quantity of seepage in an earth fill dam (Q) is governed by the hydraulic gradient acting upon the dam and the permeability of the material in which seepage takes place, in addition to the surface area (A) through which seepage occurs. When the flow is laminar, as in dams. Darcy Law governs:

$$Q = k\,h\,A$$

Here, the constant (k) is the coefficient of permeability or simply permeability of the dam material, and (h) is the hydraulic gradient across the dam, which is dimensionless. Examining this equation, it is readily inferred that for the same hydraulic gradient and surface area, seepage quantity is governed by the permeability of fill material. Permeability of the fill, however, does not depend only on the type of soil (Clay, Sand-gravel or Random Fill) but also on the degree of compaction to which this fill was subjected during construction. Since permeability of any soil cannot be made zero for any degree of compaction, it follows that seepage cannot be stopped in soil fills, but its quantity may be reduced by reducing (k). Therefore, the first two measures for seepage control is to select materials of lower permeability and to compact these materials sufficiently to reduce (k) further and not to leave such voids in the fill which may act as pipes. This can be done for homogenous dams or in zoned earth fill dams with clay cores, or by using impervious concrete or asphaltic concrete diaphragms.

Reduction of seepage to safe limits is most important for earth fill dams and this may require additional measures to be taken to make sure that failure conditions do not develop during the operational life of the dam. This is based upon the recognition of the inevitable existence of cracks of various forms or weak zones within the dam which may have formed during construction or service life of such dams.

There are currently three basic methods for controlling seepage. They are:

1. Employing methods to reduce the quantity of seepage such as clay cores, diaphragms in dam body, diaphragms and grout curtains in the foundations.
2. Using filters to prevent soil particle movement under the action of seepage force. Within this category falls using filter zones adjacent to clay cores. Filter fill benches or filter toes in the downstream side of the dam.
3. Avoid as much as possible the use of diffusive clays or suffusive materials in dam construction, otherwise corrective measures should be applied.
4. Using drainage methods to relieve seepage pressures and to collect seepage and convey it safely out, such as drainage chimneys and drainage blankets, or relief wells in artesian conditions within the foundation. Frequently, these methods are used in combination.
5. Particular attention must be made to avoid situations inducing differential settlement, which creates tension zones followed by cracking. Control over construction procedures must be exercised to avoid hydraulic fracture, arching, or having low density compacted fills adjacent to conduits or rock projections and corners in dam cutoff trenches; all such situations may create separation surfaces or pipes leading to erosion under hydraulic head.
6. In concrete gravity dams, the joints between concrete blocks in any section of the dam must be made tight enough against the penetration of water, which under freezing and thawing conditions can open up or enlarge existing thermal cracks allowing full uplift pressures developing under or within these blocks threating the stability of that part of the dam. In the case of Bouzey Masonry Dam described in paragraph 4, such seepage had caused the deterioration of the bonding material between the masonry blocks which led to the complete destruction of this dam.

7. Drainage of foundation in artesian conditions may be done properly by drainage blankets downstream of earth dams which also apply additional weight to stabilize the excess uplift pressure and prevent sand boils. Relieving such pressure may be done successfully by relief wells. Similarly, such wells can be used successfully at the downstream of concrete gravity dams which in most cases have also drainage curtains under the body of the dam, where seepage water is collected in drainage gallery(ies). In both earthfill and concrete dams grout curtains constructed to a suitable depth serve the purpose of reducing hydraulic head and thus uplift and seepage in these foundations. Some sources, however, have cast doubts on the efficiency of grout curtains in reducing uplift pressure.

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Dam Safety: Technical Problems of Ageing Concrete Dams

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Abstract

Concrete dams age as all man-made structures. Being subject to various external influences and internal reactions their ability to withstand them diminishes with time. Description of these factors are given here. The manifestations of aging signs are cracking, expansion, spalling and scaling of concrete surfaces, change of color and efflorescence, gelatinous discharge, crumbling of concrete masses, in addition to abrasion and cavitation of surfaces. The mechanisms of the actions leading to these damages are described and supported by many examples and case studies. The general conclusion drawn is that nothing can be made to extend the lives of old dam indefinitely, but a lot can be done to elongate their useful service with repair and upgrading works until technical considerations prove them unfeasible or their cost become prohibitive.

Keywords: Dam Aging, Cracking, Expansion, Spalling, Scaling, Abrasion, Cavitation.

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1. General

Aging of concrete dams may result from both physical and chemical factors. The former relate to changes in forces acting on the structure, including those caused by temperature variations, settlement, earthquake vibration, blocked drainage, or relief wells. The latter are associated with infiltration of aggressive waters containing inorganic acids, sulfates, and certain other salts into the dam body or its foundation. Chemical reactions of these substances with constituents of concrete can result in its leaching and disintegration or expanding followed by cracking and spalling. Soft water, for example, may attack concrete causing serious damage by freezing and thawing within few years. Defective or inferior materials used in the construction of a concrete dam can result in deterioration and possible failure of the structure. Poorly bonded cement, weak aggregates, or mineral-laden water can produce low-strength concrete. Highly absorptive aggregates may be susceptible to freeze-thaw damage. Aggregate contaminated by soils, salts, mica, or organic material, also may produce substandard concrete. Concrete mixes for massive structures usually contain air-entraining agents which appreciably improve the durability of the concrete and increases resistance to freezing and thawing. Yet, distress still can occur where entrainment is insufficient or when the aggregate itself is vulnerable to freeze-thaw action. Closely spaced parallel cracks at edges of concrete blocks in gravity dams may be symptomatic of freeze-thaw expansion. Entrance of water into the cracks and subsequent freezing are likely to further the deterioration.

In summary, disintegration of concrete may be caused by freezing and thawing, thermal expansion, and contraction, or wetting and drying. Freeze-thaw effects are most likely to be found in parapets, cantilever beams, slabs, and walls of appurtenant structures. All the above reasons contribute to weakening the structure, being a concrete dam or any other concrete hydraulic structure and cause its aging. Dam settlement and cracking of aged concrete structures may be attributable also to external physical factors such as uplift, foundation displacements, ice thrust, or seismic forces. Such factors and their impacts on dams are well documented in technical literature especially in dam failures and accidents study cases over the last century. Intrinsic physical factors which play in negative fashion to dam safety are those causing wear and tear to the concrete materials of the dam and its hydraulic steel structures.

Outlet structures and spillways may suffer appreciable damage by the abrasion of debris impacting on concrete surfaces in high floods degrading these structures. In spillways or outlet works conveying high velocity flows, offsets in the conduit surfaces may cause cavitation. Moreover, vibration of structures by earthquake, water surges, or equipment operation may damage concrete. Damage due to the overstressing of a concrete dam often is identified by examination and careful observing of clues which include cracking, opening at joints or lift surfaces, seepage variations, and displacements. Erosion of concrete may be caused also by flowing ice, sediments, logs, wind, traffic, or cavitation.

One of the most common problems reported in concrete gravity dams is clogging of drainage systems. The need for regular maintenance of drains is well recognized. Obstruction of dam’s foundation drains may be attributable to various causes, including displacement, soil or rock deposits, biological growth, and leaching and deposition of chemical [1].

2. Deterioration of Concrete in Old Concrete Dams

Deterioration of many old concrete dams which have outlived their usefulness, or become safety hazard, has been attributed in many cases to the chemical reaction of the constituents of cement and aggregates making up the concrete in the presence of water over a long period of time. A typical example of such reaction is the Alkali-Aggregate Reaction (AAR). Such chemical process is evidenced in upstream movement of an arch dam crown, spalling of the concrete at extremities, and by characteristic pattern cracking and crazing of the dam face. The strength of the concrete mass may be reduced by alkali-aggregate reaction also.

Visible clues to such deterioration include:

1. Expansion.
2. Cracking of random pattern.
3. Gelatinous discharge.
4. Chalky surfaces.

Petrographic examinations of the concrete cores taken from affected structures have revealed severe fracturing and tests have also shown a strength reduction as much as 25% or more.

Expansion in the decomposing concrete can be substantial, but rates of movement usually appear to decrease as the dam increases in age. Alkali-aggregate reaction sometimes causes the disbonding of blocks at lift surfaces. Loss of strength by disbonding, and the accompanying increase in hydrostatic pressure along the lift surfaces will reduce resistance to sliding and overturning. Alkali-aggregate reaction can cause expansion of a concrete dam with consequent cracking and deterioration of valves and metalwork and possible binding of gates into their guides.

Once alkali-aggregate reaction has developed in a relatively thin concrete dam, it cannot be stopped practically by any means known. Where deterioration has progressed to a dangerously advanced stage, the effective remedies are to remove and replace the defective concrete or to build a new dam to replace the old one. Otherwise there will come a time when the dam is unable to withstand either static and/or dynamic flood induced loads, or earthquake loadings leading to catastrophic failure. As such dam weakens, the probability of an adverse response to a given load increases. The case of “Clear Creek Dam” may be cited here for illustration.

2.1 Clear Creek Dam Case

This concrete thin-arch dam was originally completed in 1915 by the United States Bureau of Reclamation on the north fork of the Tieton River in Yakima County, Washington State, refer to Figure 1. Height of the dam was 19m and its length was

123m at the crest. In 1918 it was raised by 6.4m to its present height of 25m and it was partially rebuilt in 1964. The dam suffered from a variety of deterioration mechanisms and the concrete properties diminished. Analysis of the dam conditions identified a strong probability of failure under static, hydrologic, and dynamic loading.

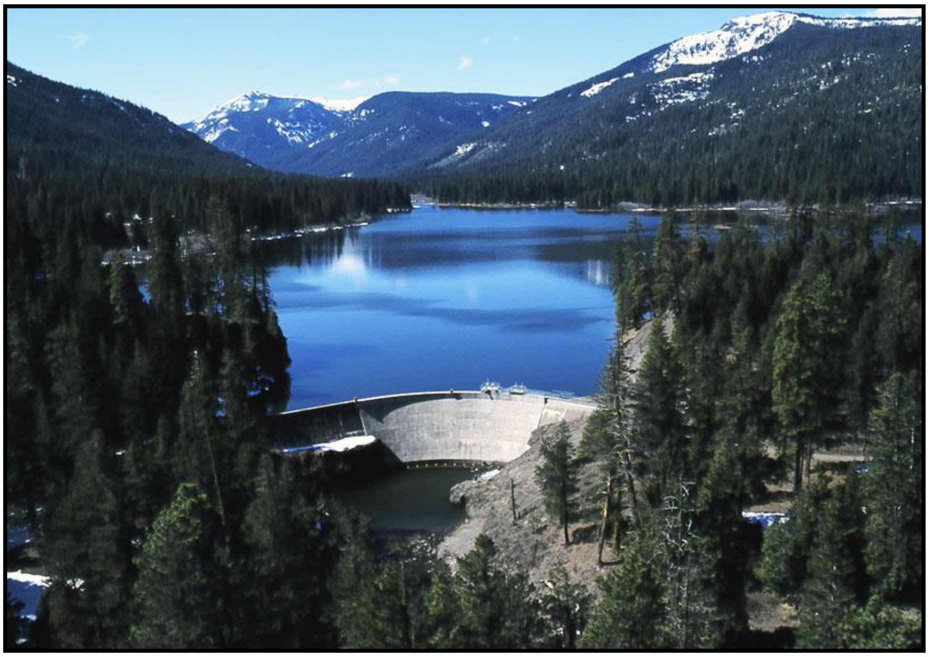

Figure 1: Clear Creek dam (Washington) downstream view.

Rehabilitation work in 1964 consisted primarily of: placing new concrete in the arch section between elevation 2991m (a.s.l.) and the crest, repairing cracks and poorly consolidated concrete with neoprene and epoxies, and installing protective wire-mesh fences from the abutments to upstream areas. Resulting from investigations conducted in 1987-1989, the Bureau of Reclamation concluded there were horizontal bands of deteriorated concrete in the section replaced in 1964. These conditions could result in sudden failure of the dam, so early in 1990 the water level of Clear Lake was immediately lowered to reduce the likelihood of dam failure to less than 5% of its design capacity by drilling two holes through the dam to further drain the lake to a point that only 230 acre-feet (283,700m^3) could be stored out of the total original capacity of 5,300 acre feet (6,500,000m^3). Reconstruction works were accomplished and consisted of converting the dam into a gravity structure by buttressing the arch with a new concrete section immediately downstream. A new 48-inch outlet conduit was installed near the elevation of the breach (2,956.5 feet) to permit discharge whenever the lake was below the spillway crest. Clear Lake which has a water surface area of 260 acres (110ha) is used now primarily for recreation including boating and fishing and most of the discharge is through the spillway [2] and [3].

3. Concrete Deterioration Mechanisms by Chemical Agents

Concrete deterioration is a progressive reduction in properties, which with the passage of time may ultimately make concrete no longer serviceable for its intended use.

This can result from:

i. Physical “removal” of materials from the surface of the structure which may be caused from erosion by debris or cavitation by high velocity flow, leading to a reduced cross section.
ii. An internal change in strength, modulus of elasticity, Poisson’s ratio, or density that reduces its overall structural load-carrying capacity.
iii. Surface deterioration and loss of material may be caused by freezing and thawing (F/T) of concrete which leads to a reduced cross section of a concrete dam. The reduced cross section increases the stresses of the remaining section proportionately to the amount of material removed.

Internal change in strength, and other mentioned properties that reduces overall structural load-carrying capacity of a concrete structure can be caused by Alkali-Aggregate Reactivity (AAR) or Sulfate Attack in processes which can take long time and their adverse impacts appear after many years of service. Concrete dams and concrete ancillary structures which are expected to have a life span much longer than their assumed life may show signs of such deterioration at late age in a similar way to human beings when potential sources of weakness become active leading to death.

4. Alkali-aggregate reaction (AAR)

This is a reaction in concrete between the alkali hydroxides, which originate mainly from Portland cement, and certain types of aggregate both used in the construction of the structure. Two types of (AAR) are currently recognized; these are alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). As the names imply, these types of reaction differ in that they involve reactions with either siliceous or carbonate types in the aggregates which lead to volume change and may reduce the strength and modulus of elasticity of the entire structure. Swelling and cracking of concrete outlets or spillways caused by ASR or ACR lead to reduced structural performance, and the cracking may accelerate other deterioration mechanisms, such as freezing and thawing (F/T) deterioration.

Problems due to ASR were first identified in USA in the State of California in the 1930s and reported by Thomas Stanton of the California State Division of Highways in 1940. Stanton’s studies demonstrated that the expansion of mortar bars was influenced by the alkali content of the cement, the type and amount of the reactive silica in the aggregate, the availability of moisture, and temperature. He further showed that expansion was negligible when the alkali content of the cement was below 0.60% Na_2Oe and that expansion could be reduced by pozzolans, thus setting the groundwork for preventive measures. Subsequent to Stanton's discovery ASR was diagnosed as the cause of abnormal cracking in a number of dams operated by the U.S. Bureau of Reclamation, such as the Parker Dam in Arizona. So, a number of agencies in USA initiated studies on ASR in the 1940s, for example; the Army Corps of Engineers, Bureau of Public Roads, Portland Cement Association. Other countries followed such as Denmark and Australia. ASR is now recognized as a major cause of concrete deterioration in dams in the world that has incidents in many States in USA and also in numerous countries worldwide [4].

Stanton studies in 1940 opened the way to the use of pozzolans for reducing the harmful effects of ASR and this was first put into practice in the same decade when calcined clay was used to prevent it in the Davis Dam constructed between 1942 and 1950. The reaction was implicated as the cause of cracking in the Parker Dam, which was completed shortly before construction began on the Davis Dam and is located 141km upstream of Davis Dam on the Colorado River. Ten years after Stanton's (1940) discovery the potential for using fly ash and slag for controlling expansion was first documented, and it is now widely accepted that supplementary cementing materials are an effective means for controlling ASR expansion.

History of several US Bureau of Reclamation old structures shows that many of them experienced damage due to this reaction. Beginning in the mid-1920s. The Bureau did not fully understand the causes of the deterioration at the time. Some of the notable dams suffering from it include; American Falls (Idaho), Owyhee (Oregon), Seminoe (Wyoming), Friant (California), Parker (Arizona), and Stewart Mountain (Arizona). Typical deterioration included swelling and cracking of the concrete, accompanied by a decrease in strength and modulus of elasticity. The cracking also provides avenues for moisture to enter the concrete and contribute to accelerated (F/T) attack in cold climates.

Methods to prevent ASR were developed by 1942 which included identifying potentially reactive aggregates using petrographic techniques, limiting their use, and specifying low-alkali cements and pozzolans [5].

For further elaboration on ASR problems on dams, two study cases are considered here for further explanation:

i. The American Falls Dam (Idaho).
ii. Seminoe Dam (Wyoming).
iii. Pracana Dam in Portugal.

4.1 The American Falls Dam (Idaho), Study Case

This dam, which was completed in 1927, is a 94-foot-high composite concrete and earth gravity-type dam on the Snake River near American Falls, Idaho, which holds a storage capacity of 1,700,000 acre-feet ($2,1x10^9 m^3$).

Principle benefits of the dam include irrigation, power generation, flood control, fish and wildlife resources, and recreation. During the years following construction, signs of distress appeared on the dam. Core-drilling program in the early 1960s revealed that the concrete in portions of the dam was in a relatively advanced stage of deterioration due to a chemical reaction between alkalis in the cement and the aggregate (ASR). This type of reaction, unknown at the time of construction, resulted in a significant loss in strength and durability, threatening the competence of the dam and resulting in filling restriction that reduced the storage capacity of the reservoir to about 66% of its maximum design capacity. In 1973, works for construction of a new dam started and it was completed in 1978, while the original structure was demolished. Later on, similar problems appeared in the new dam.

A value engineering study that was initiated in 2015 recommended the following corrective actions:

a. Removal and replacement of 6 inches of concrete on the spillway face and stilling basin floor.
b. Repair of concrete on the upper spillway gate operator's decks and complete replacement of the spillway adits (access entryways).

The study stressed the need for improving the structural integrity of the spillway to avoid further deterioration which could lead to serious structural deficiencies, and provide a more feasible means of access for future maintenance activities. The cracked and damaged state of concrete on the spillway, spillway gate operating decks, downstream dam face concrete, and stilling basin floor structures created strong need for action. These concrete components of the dam structure were exhibiting significant deterioration, cracking, and spalling, and required repair. Minor repairs, however, were already completed to the spillway face throughout its lifetime, including an overlay of the stilling basin floor that was completed in 1978 to repair damaged concrete after the initial spill season. A new study was completed in May 2019 which endorsed the 2015 findings and included an environmental impact assessment study.

The planned construction activities will take place on the downstream portion of the dam. This project will address the need for replacement and repair of deteriorating concrete on the spillway, spillway gate operators' decks, downstream dam face, adits, and stilling basin structures, which have experienced degradation over 40 years of service, refer to Figures 2 and 3.

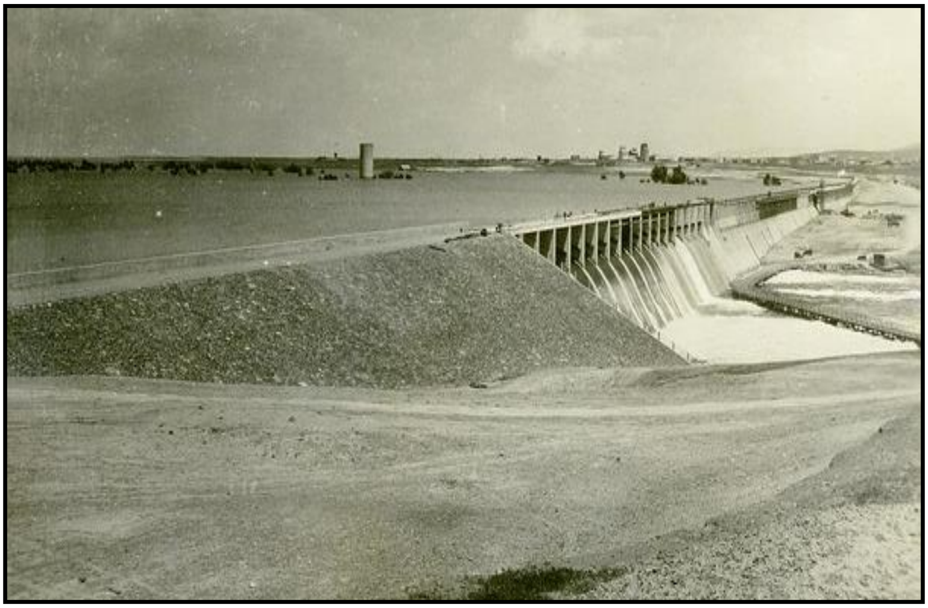

Figure 2: View of the old original American Fall Dam (1927).

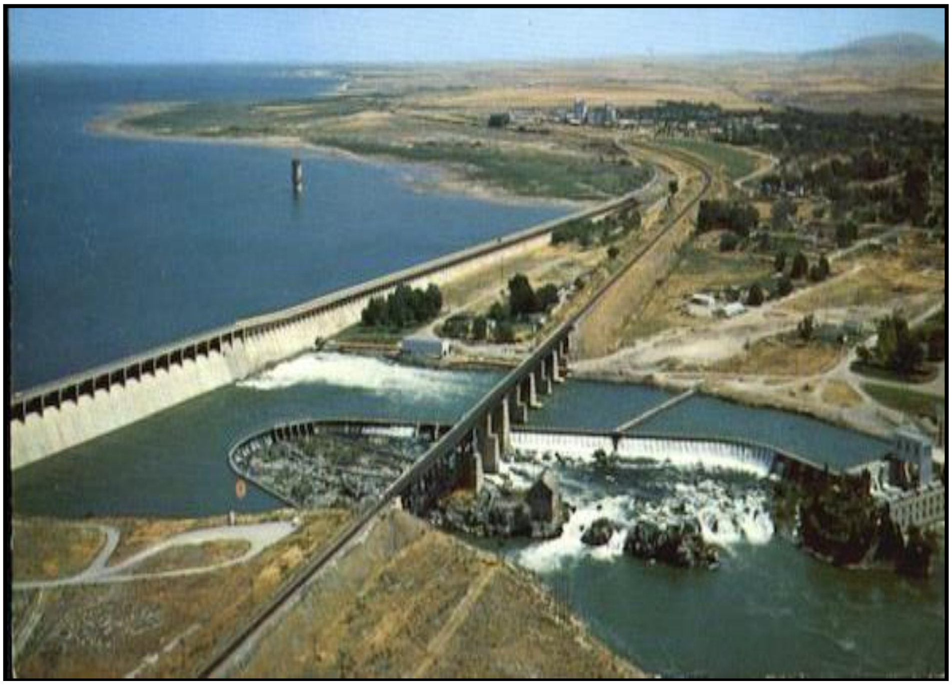

Figure 3: Aerial view of American Fall Dam after 1978.

Construction activities will consist of the demolition, cutting, removal, and replacement of existing damaged concrete and reinforcing of these components, and replacement or modification of an existing drain grate in the stilling basin. Construction will take place from June to November 2020 and mid-July to November 2021. Examples of the current state of deterioration in this dam are pictured and presented in Figures 4 , 5 and 6 [6].

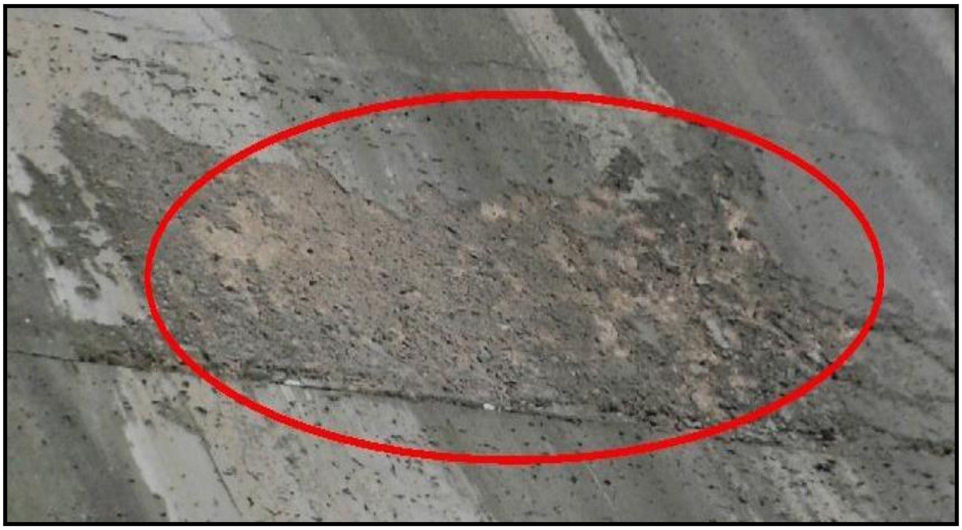

Figure 4: Disintegrating surface concrete observed on dam face.

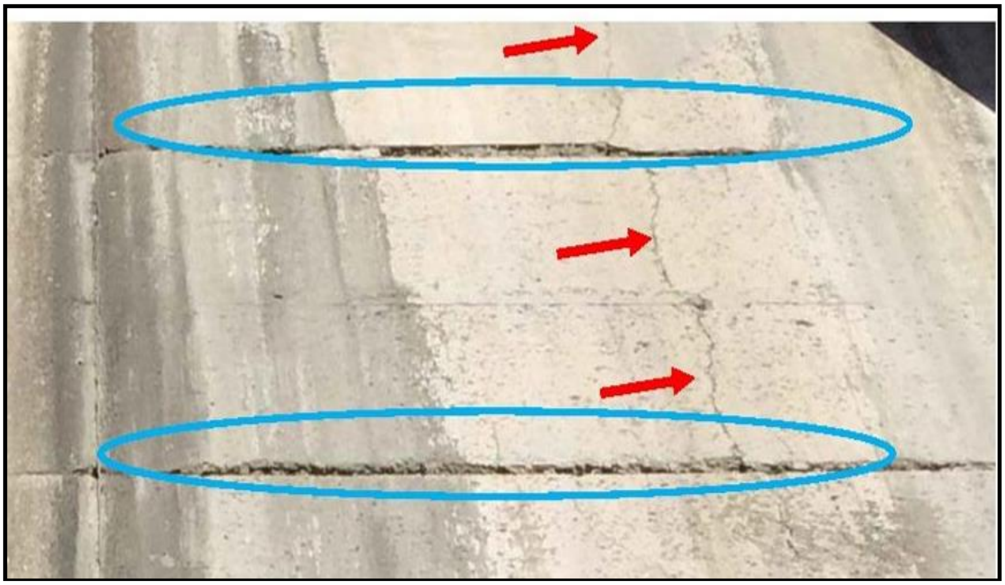

Figure 5: Deterioration of concrete joints observed on the spillway face.

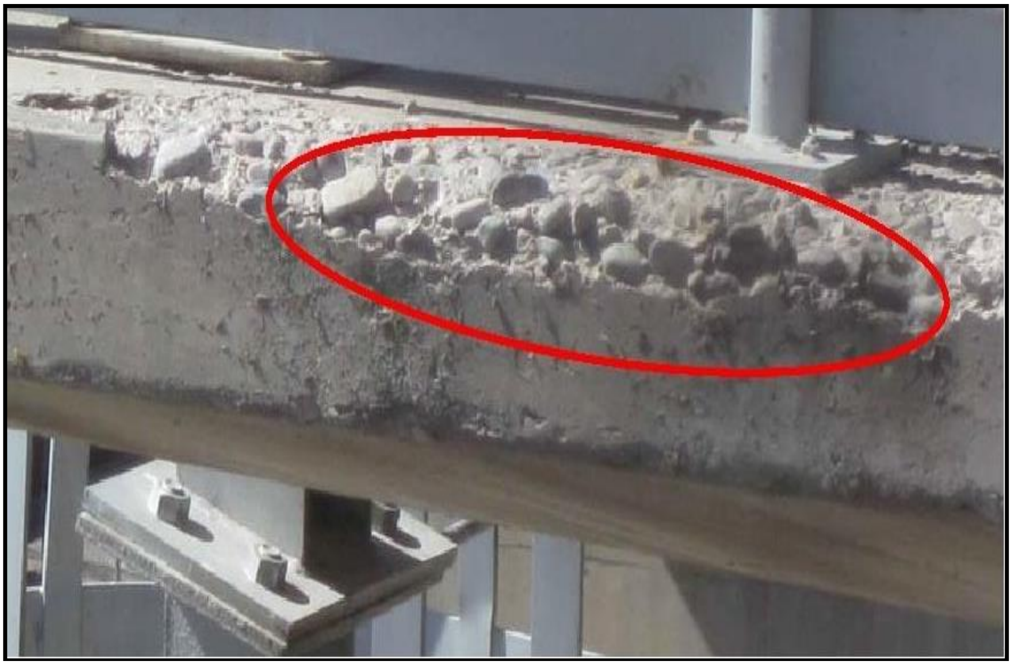

Figure 6: Structural deterioration of concrete on spillway gate operator deck. Extensive exposed aggregate is apparent.

4.2 The Seminoe Dam (Wyoming) Case

Seminoe Dam is a concrete thick-arch dam on the North Platte River in the U.S State of Wyoming. The dam stores water for irrigation and hydroelectric generation, and is owned and operated by the U.S. Bureau of Reclamation.

The 295-foot (90m) dam was constructed in 1939 and forms Seminoe Reservoir, which covers more than 20,000 acres (8,100 ha) when full. The dam is exposed to severe winter conditions, fairly rapid and extreme temperature changes, and frequent freeze-thaw cycles. A few years after construction some cracking, and deterioration of the concrete was observed. This was at a time before the discovery that low alkali cement, air entraining admixtures, and other beneficial modifications to concrete could ameliorate deleterious chemical and physical deterioration.

Downstream and upstream views of the dam are shown in Figures 7 and 8. Cracking and deterioration of concrete was chiefly observed along the upper parapet walls and power house walls. Although since the 1950s, ASR was suspected of causing some of the cracking visible at the top of the dam, ASR did not appear to be a major concern. In 1951, a petrographic examination of the concrete revealed indications of ASR along with freeze-thaw deterioration. It was not certain whether ASR or freeze-thaw deterioration was the main cause for this deterioration. The examination revealed the presence of about 4.5% reactive particles, chiefly cherts, andesites, and rhyolites, which were judged to be only marginally deleterious.

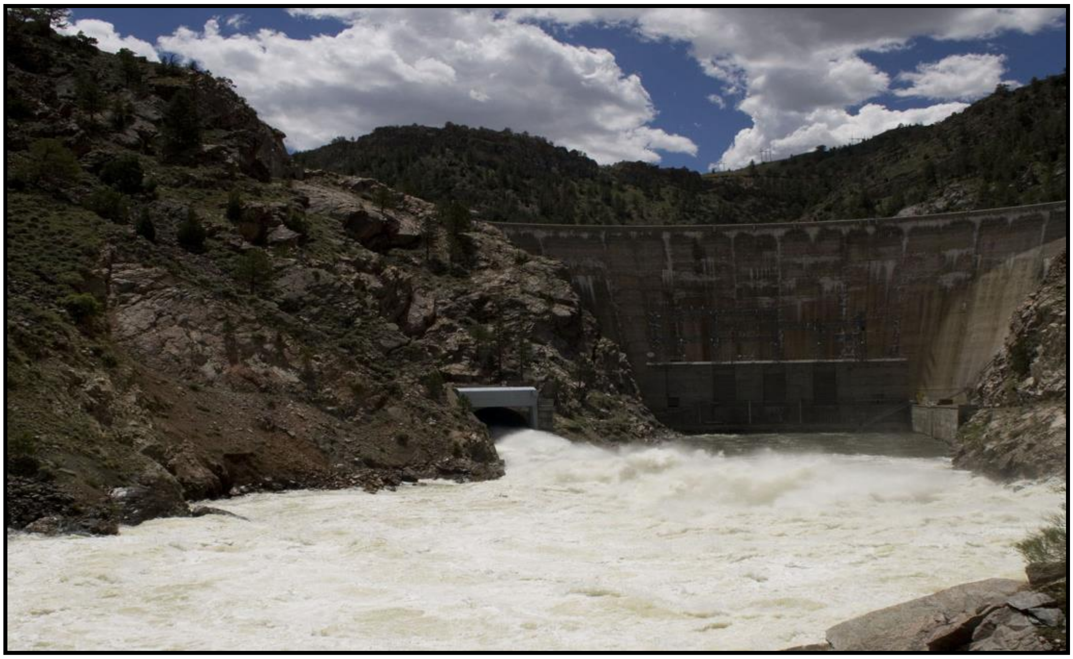

Figure 7: Downstream view of Seminoe Dam.

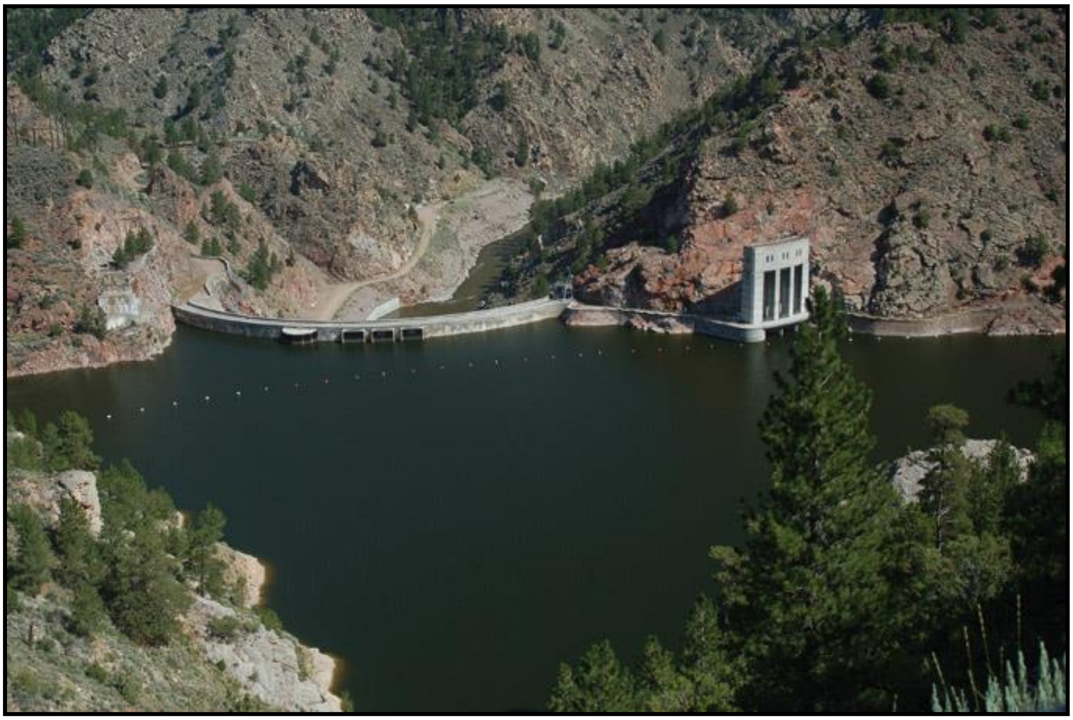

Figure 8: Upstream view of Seminoe Dam.

Part of the reason of this uncertainty was that petrographic examination indicated that a potentially deleterious reaction was unlikely based on the composition of the aggregate particles and tests for reactivity. At this time, most experience with ASR indicated the reaction showed up fairly early in the life of the structure and was fairly easy to identify, but these experiences were with more reactive aggregates with structures in warmer climates.

In a 1970’s, petrographic examination and on concrete cores indicated extensive damage to the upper 5 feet with minimal to moderate damage to about 20 feet. In a 1980’s petrographic examination of cores testing indicated extensive damage to the upper 8 feet of concrete and minimal to moderate damage below 8 feet. In the 1990’s several subsequent petrographic examinations showed more indications of alkali aggregate reaction and that the alkali aggregate reaction was continuing. Later examinations indicated the reaction was a slowly reactive form of ASR involving quartzite containing strained quartz, which is known to be a slowly reactive form of silica. In these examinations areas of extensive damage were observed to the upper 18 feet of the dam.

In 2013, a coring program provided fresh cores drilled from five vertically oriented drill holes on the crest. Selected intact core fragments were tested for physical properties and strength conditions, as well as, petrographic analysis to determine the current condition of the concrete. In this 2013 program, evidence of ASR was observed in cores up to 75 feet depth [7].

Some concrete repair and stop log replacement project was recently completed at Seminoe Dam. The river outlet works and stilling basin at Seminoe Power Plant was identified to have deficiencies in the concrete located around the outlet works and the tailrace deck. The stop logs were constructed as part of the original Seminoe Dam and Power Plant in 1939 and over 81 years of wear and tear had taken their toll. They no longer provided an adequate seal to allow periodic maintenance of the power plant draft tubes and river outlet gates. The tailrace deck also had deteriorated concrete and the stairway leading to the tailrace deck did not meet accepted standards.

The repair works were started on February 8, 2016, beginning with construction of a cofferdam to protect the work site from flows passing through the power plant. Upon completion of the cofferdam, the work area between the upstream face of the dam and the cofferdam was dewatered. Areas around the damaged concrete that required repair were removed using chipping hammers to the required design depth. New steel reinforcement and concrete was placed in the repair zones. Cracks were sealed using a pressurized epoxy grout system. The contractor completed all of the concrete repairs in the tailrace area protected by the cofferdam by March 23, 2016. The contractor removed the cofferdam from the river by April 6, to allow for potential water releases through the river outlet works.

The tailrace deck was resurfaced by saw cutting the surface, using jack hammers to remove deteriorated concrete, and then adding a new layer of fresh concrete. Photographs of the performance of some of the described works are shown in Figures 9 and 10 [8].

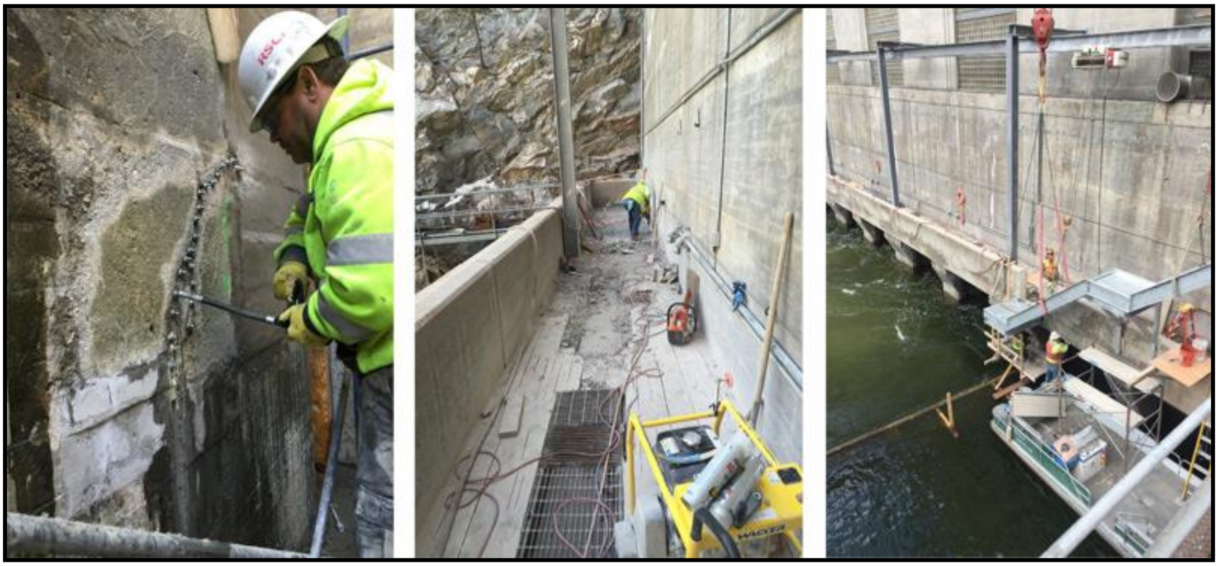

Figure 9: Left: Injecting epoxy to repair crack. Center: Concrete removal on tailrace deck. Right: Installing new stairway.

A new study by Colorado University at Boulder is underway now, on behalf of the Bureau of Reclamation to study the long term assessment of dams suffering from Alkali Aggregate Reaction. This reflects the concerns of the Bureau over the safety of their great number aging concrete dams, and even ancillary concrete works of their old embankments dams due to this reaction. This study may give some indications on the overall safety of these dams and can open the way to decommissioning of some [9]. Countries other than USA have also suffered of the same problem in their dams, but the discovery of this phenomenon and the extensive work for remedial action was pioneered in USA with extensive volume of research work and documentation.

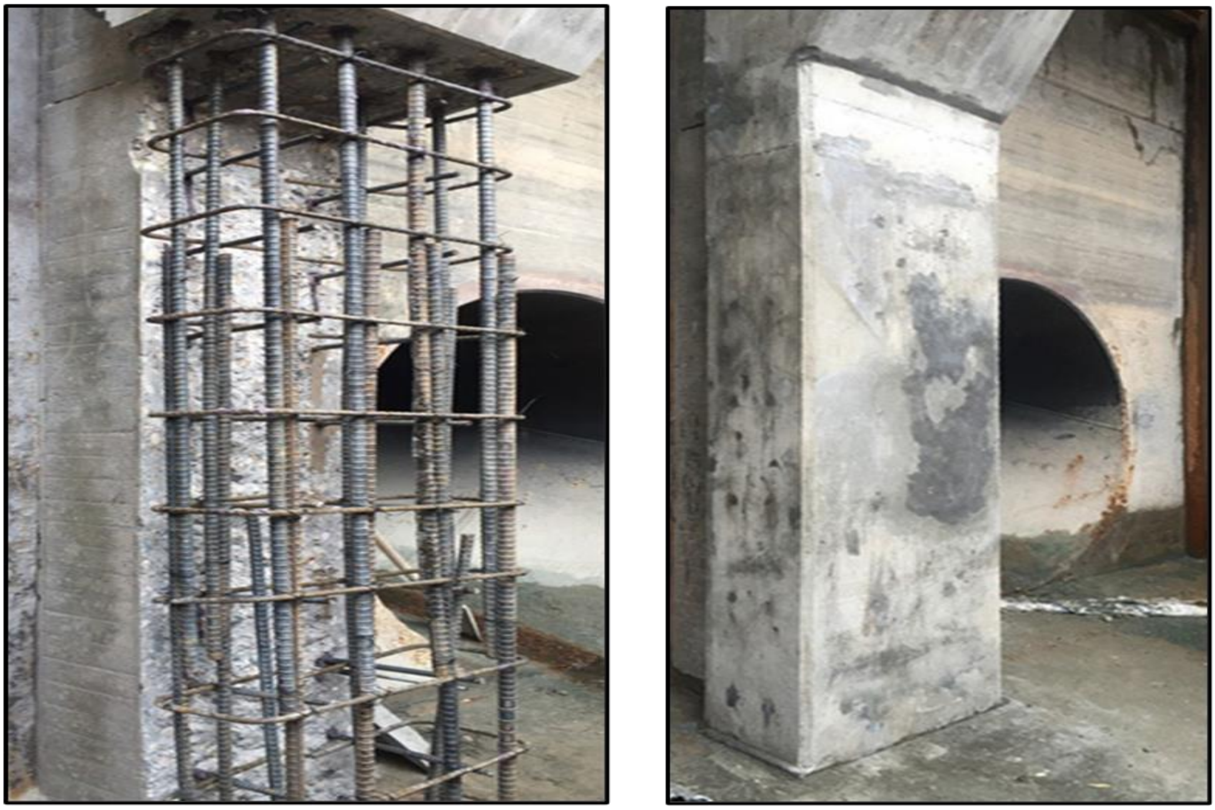

Figure 10: Repairs in river outlet works center pier showing reinforcing bars and the finished shape.

4.3 The Pracana Dam Case

This is one more case in Portugal this time which can be cited in relation to a novel solution for rehabilitation of a dam suffering from AAR which is that of Pracana Dam. The dam, a 65 m high concrete buttress dam in a seismic region of Portugal, was constructed between 1948 and 1951, refer to Figure 11. In 1980, the dam was taken out of operation to undertake a thorough investigation of the deterioration related to the continuous concrete expansion phenomena. Investigations ascertained the presence of a secondary alkali-aggregate reaction which would be further activated by infiltration of water from the reservoir, creating a critical scenario in respect to sliding conditions along horizontal cracks. Installation of a drained waterproofing liner on the upstream face was deemed necessary to stop water infiltration feeding this reaction and to avoid the possibility of water exerting uplift in the horizontal cracks, especially in the case of a seismic event. The installation of PVC geomembrane was chosen as protective solution and it was completed in 1992. The exposed drained geomembrane system was installed in the dry season of 1992, in 5 months, concurrent with major rehabilitation works including construction of a new foundation beam and grout curtain, two sets of concrete struts on the downstream face, local grouting of larger cracks and mass grouting of smaller cracks, construction of a new spillway and a new water intake. Since 1992, the behavior of the geocomposite waterproofing system has been monitored in respect of leakage and its capability of dehydrating the dam, reducing the water content feeding this harmful reaction, and to avoid the possibility of water exerting uplift in the horizontal cracks, especially in the case of a seismic event. Two photographs showing the dam during and after rehabilitation work are presented in Figure 11 [10].

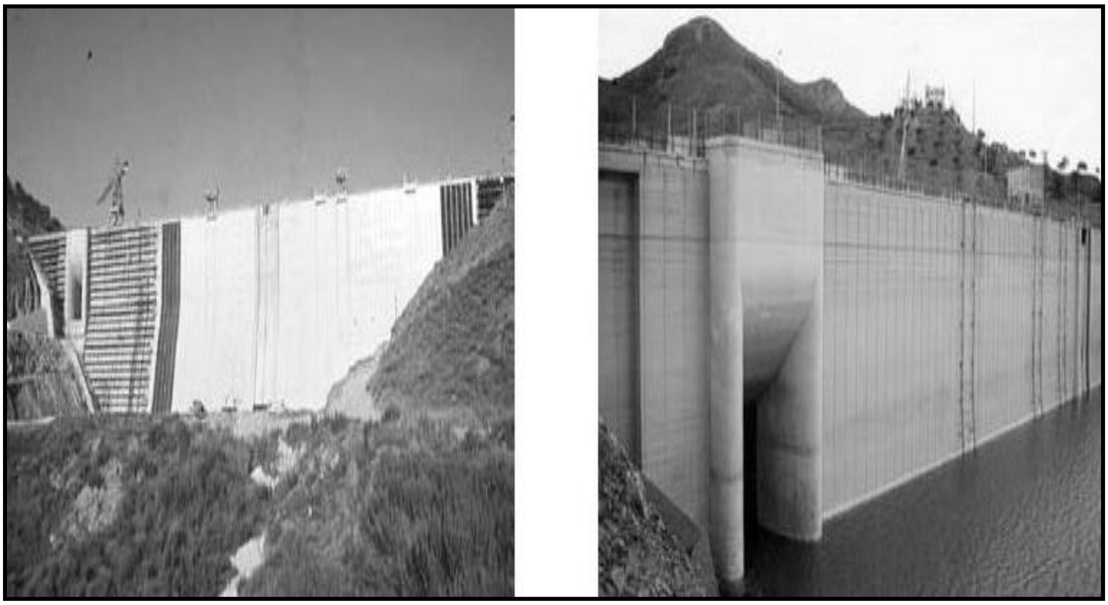

Figure 11: Pracana Dam affected by Alkali Aggregate Reaction during and after rehabilitation by water proofing.

In our opinion, this solution remains to prove its credibility after many more years of operation on; first, the durability of the material itself; second the possibility of its use in other situations in dams, and third, the economic feasibility of such installation.

An article published by the Canadian Journal of Civil Engineering in 2011 with the title “Alkali-aggregate reaction in concrete: a review of basic concepts and engineering implications” gives a comprehensive analysis of the Alkali-aggregate reaction problems in concrete structures including the basic concepts of the reaction and expansion mechanism, conditions conducive to the development and sustainability of the reaction in concrete, in addition to the selection of preventive measures and management of structures affected by it [11].

Reference to this paper help the understanding and early discovery of the phenomenon in concrete dams, and on required actions of protection to elongate serviceability of dam under question. A presentation in conference, given by one of the authors of this paper gives more details with explanations and photographs illustrating examples of damage in concrete structures including dams in many countries [12]. More on this subject may also be found in Portland Cement Association publication by (James A. Farny and Beatrix Kerkhoff "Diagnosis and Control of Alkali-Aggregate Reactions in Concrete") [13].

In summary, alkali-aggregate reaction on gravity dams and ancillary structures, can sometimes cause the disbonding of blocks at lift surfaces. Loss of strength by disbonding, and the accompanying increase in hydrostatic pressure along the lift surfaces will reduce resistance to sliding and overturning. Moreover, alkali-aggregate reaction can cause expansion of a concrete dam with consequent cracking and deterioration and possible restriction of gate movement and jamming of valves, and deformation of metalwork.

5. Sulfate Attack on Concrete

Concrete in dams may be the object of another form of harmful reaction caused by sulfate attack. It is a form of chemical degradation of cement paste caused by high concentrations of sulfates in wet soils or ambient water. Sulfate attack is caused by chemical interactions between sulfate ions and constituents of the cement paste and/or salts in the concrete mix fine aggregate which is an internal sulfate attack (ISA) or from sulfates dissolved in water in contact with the structure, their origin is external and therefore provoke the so-called external sulfate attack (ESA). The disintegration of concrete appears to be caused by chemical reactions with cement hydration products and the formation of a secondary compound called Ettringite, accompanied by a large volumetric-expansion and causing cracking of the concrete and its disintegration [14].

Normally, cement contains about 8% of (Calcium - Aluminum - Sulfate i.e. C3A) known as ringite. But it is usual in the production of cement to add to the other constituents a proportion of gypsum (about 6%) to prevent excessive solidification when water is added to the other constituents when concrete is mixed. At the mixing process of concrete before use, adding water results in formation of calcium aluminate hydrate called Ettringite which contains 26 molecules of water. The formation of Ettringite causes temporary volume expansion in the concrete mixed but it converts back within the first 24 hours to less voluminous calcium aluminate monosulfate. If, however, hard concrete with high percentage of calcium aluminate monosulfate is exposed to water in damp soil or as solution, then this water can react with the excess calcium aluminate monosulfate converting it back to ringite and then to Ettringite after uptake of more water. With this continuous up-taking of water, the increasing expansion gives rise to the damaging impacts observed on concrete.

Most prominent sulfates among aggressive substances which affect hydraulic concrete structures are the sulfates of sodium, magnesium, and calcium. These salts, which are known as white alkali, are frequently encountered in the alkali soils and ground waters of many countries around the world. The stronger the concentration of these salts the more active is this corrosion of concrete. Sulfate solutions increase in strength in dry seasons when dilution is at a minimum. To avoid such harmful results in dam construction careful selection of aggregates and cements should be exercised, and in many cases, it is advisable to use sulfate resisting low alkali cements for this purpose.

An example of damage caused by long time action of sulfate attack on hydraulic structure is shown in Figure 12 [15]. From the United States Bureau of Reclamation literature, examples of sulfate attack in spillways are documented, such as the tests performed on concrete cores from the south spillway at Guernsey Dam (Wyoming), and Alcova Dam, Kendrick Project, (Wyoming). Both dams are located in areas which are now known to have potential for high concentrations of sulfates and were built before the first sulfate resisting cements were used by the Bureau [16].

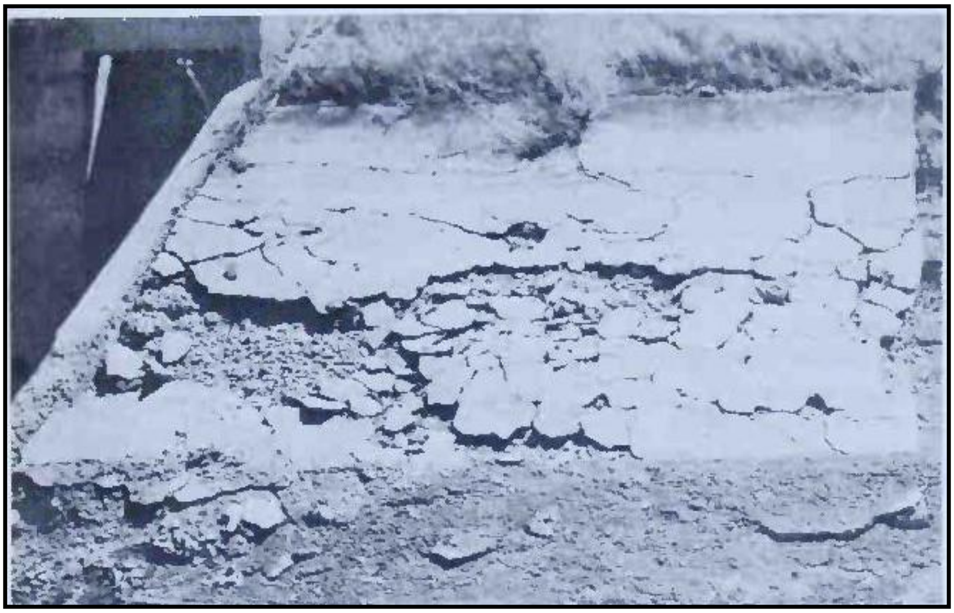

Figure 12: Disintegration of concrete caused by sulfate attack.

After first reporting of sulfate attacks on concrete, the phenomenon has been widely-studied and well-documented as a type of degradation in hydraulic structures.

As already been explained; sulfate ions dissolved in water in contact with the structure provokes the so-called (ESA) and sulfates ions present in the concrete itself is called (ISA). The internal presence of sulfates can occur when the concrete has an excess gypsum used as setting retardant in the cement hydration process or, in the concrete aggregates used which contain gypsum, iron sulfides or other salts together with the sulfates. One example of Sulfate Attack is the case of Mequinenza dam (Spain), explained in the following paragraph.

5.1 The Mequinenza Dam Study Case

The dam at Mequinenza is a gravity dam located in the province of Zaragoza (Spain), and is located on the Ebro River. The maximum dam height is 79m and its crest length is 461m. The total dam body volume is ($1.1\times10^{6}m^{3}$). The reservoir was first filled in December 1965, although its level was later lowered and filled again in 1969, refer to Figure 13.

The Mequinenza dam was built upon materials in which chemical sediments (limestone and gypsum) are predominant over detrital ones.

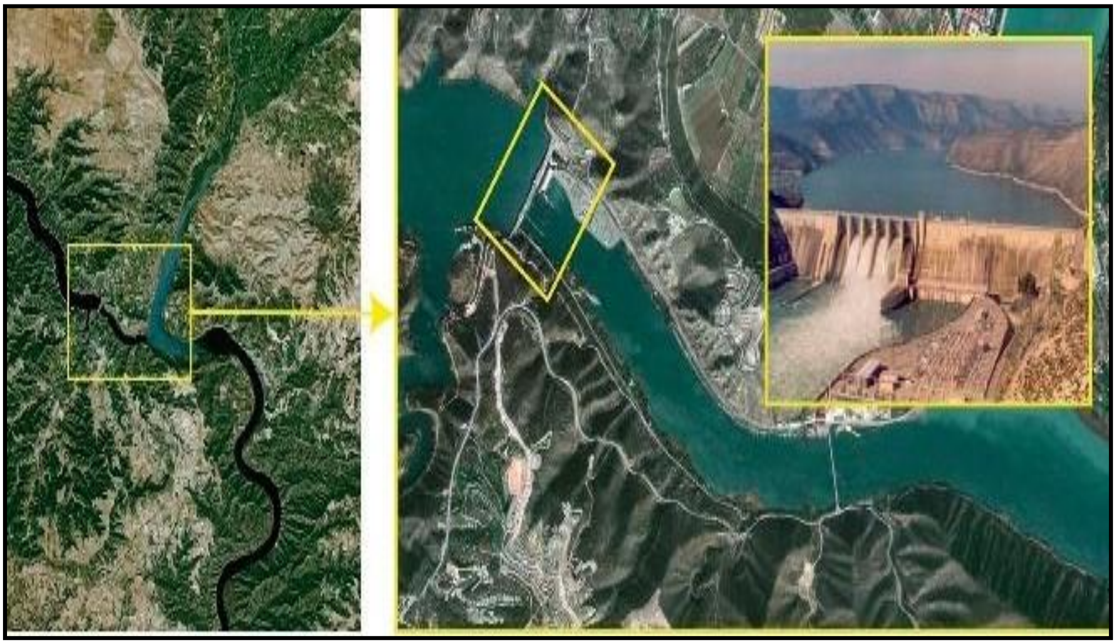

Figure 13: Left: Location of Mequinenza dam. Right: Downstream face of the dam.

As it happens in most of the Ebro Basin (except the strips near the ranges that surround it), the strata from the Tertiary period remain practically horizontal, just as they were deposited. The aforementioned carbonate domains of these units include coal lignite layers in the Mequinenza mining district. These are groups of small layers with thickness of 20–80cm characterized by their sub horizontal position and great continuity. The lignites at Mequinenza are high-sulfur content coals. Some reports reported an average sulfur content of 13.35% (expressed as total sulfur), similar to the 13.6% found in another source. Other source claims dry-sample sulfur contents for lignites ranging from 7.6% to 11.4%. Sulfate attack problems appeared on various degraded parts of the dam. Efflorescence samples were collected, and concrete core were extracted from all concrete blocks and auxiliary elements of the dam especially the most degraded galleries. These galleries are Gallery 22m and Gallery 81m located in an area of the abutments in contact with the terrain where lignites exists, refer to Figure 14.

Aggregates from the gravel and from the surroundings of the dam and lignites from the old mine were also collected. Examination of aggregates from the dam concretes revealed that it was composed of rounded gravel, this gravel was extracted from the surrounding area, in this case the Lower Pleistocene terraces. The dam was completed in 1964 and there is no precise reference to the type of cement used in the concrete in construction. No mention is made in the Dam Book of the concrete additives that could have been used. The concrete core samples were stained, and tests showed neither sulfates nor gypsum have been found, together with the limestone nature of aggregates, which rules out the possibility of an internal sulfate attack in concrete. Most of the concrete core samples did not show signs of degradation owing to Alkali Aggregate Reaction except some rims in aggregates and white gel, both typical signs of an Alkali Aggregate Reaction. Few samples analyzed by means of SEM-EDX showed AAR gel as well as secondary Ettringite filling the concrete pores.

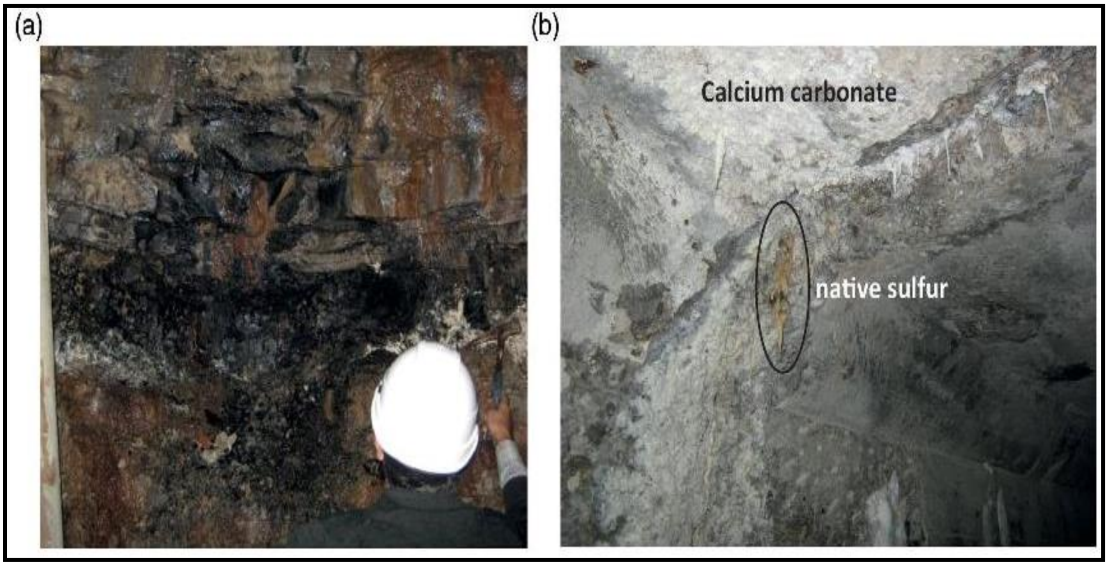

Figure 14: a) Lignite horizons in rock present in the abutment at gallery 22 m level. b) Part of wall and ceiling of gallery 22.

However, these degradation products are not fully extended in the concrete matrix and are located in a few pores. But, Thaumasite Sulfate Attack (TSA) was evident in gallery 81m indicating signs of a severe concrete degradation in the chutes through which the filtered water flows in the gallery and, to a lesser extent, in the walls, refer to Figure 15. Thaumasite is a silicate mineral with chemical formula $Ca_3Si(OH)_6(CO_3)(SO_4)\cdot 12H_2O$.

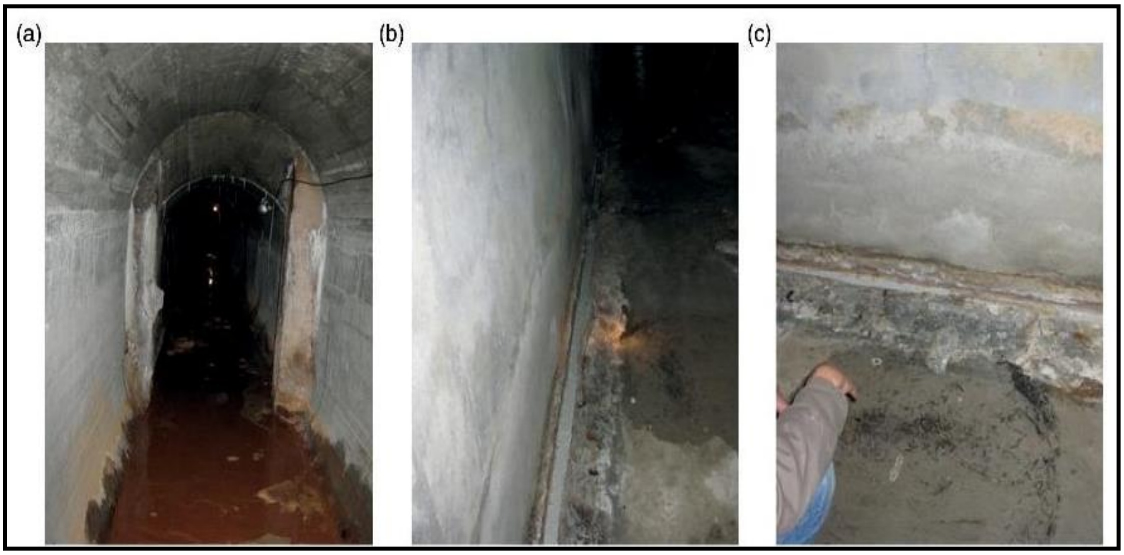

Figure 15: a) Brownish waters in the floor of gallery 81m.
b) and c) Photos of the wall and chute where water is flowing in gallery 81 m.

Thaumasite sulfate attack (TSA) is a special form of sulfate attack typically occurring due to the availability of carbonates in the cementitious matrix with abundance of moisture and a prevailing low temperature. It transforms concrete into a non-cohesive mass without any binding or load carrying capacity.

The sulfate concentration in water, analyzed by atomic absorption, was 1528ppm. More specifically, at least the 5 first centimeters of concrete in the chute crumbles in the hand. In this zone samples were extracted every 2 meters and the analysis of concrete shows that degradation was caused by a sulfate attack in which thaumasite has formed.

Expansive reactions in the concrete of certain parts located near the abutments of the two galleries have been observed as a consequence of an external sulfate attack due to the sulfur compounds contained in the lignites that are present on the surrounding terrain. The Mequinenza dam concretes are made with rolling aggregates which mostly have a carbonate nature. No iron sulfides or other sulfur compounds appeared in those materials, which is one reason why an internal sulfate attack (ISA) can be ruled out.

Some other authors have already explained that the highest degradation of concrete occurs in areas which have been in permanent contact with water, more specifically in the chutes that act as drains in one of the galleries and in the walls where water seepage appear. A large part of the concrete in those chutes has become mass without any cohesion during a sulfate attack process leading to (TSA) formation in a case of External Sulfate Attack [17].

Further reading on sulfate reaction impact on concrete dams can be made by referring to the case of Lijiaxia Concrete Dam Gallery (China) [18]. One more paper can be also referred to on three Spanish dams plagued by sulfate reactions, namely; Torán, Graus and Tavascán dams and under the title “Evaluation of the behavior of concrete gravity dams suffering from internal sulfate attack” [19].

6. Concrete Deterioration Mechanisms Caused by Physical Agents

These mechanisms may be differentiated in those caused by temperature variations, settlement, sediments and debris, earthquake vibration, and high velocity flow and appear in one of the following forms.

6.1 Abrasion of Concrete

Concrete Dams and concrete ancillary works in earth fill dams are subject to damage due to physical forces acting on the dam. One of these physical factors causing wear and tear to concrete surfaces in the waterways is abrasion. Abrasion results from both concrete with low strength and poor aggregates, and by design related problems, so that rocks and sediments carried by the strong water current to the downstream are sweep back into spillway and outlet works stilling basins resulting in particles abrading the surface in a roller-mill fashion. The hydraulic jump sections of those stilling basins where turbulent flow conditions occur, are particularly vulnerable to abrasion damage. Even the best concrete cannot withstand this wearing action for very long time and such damage results in disintegration of the material exposed to the abrasion mechanism. Erosion damage can be very severe but generally would not cause total failure. It may, however, impair the function of the structure and require intensive costly repair works, while such repair works interrupt its operation causing additional economic losses. Figure 16 shows the abrasion erosion damage that occurred to the dentate, walls, and floor areas of the Yellowtail Afterbay Dam in USA sluiceway stilling basin. Characteristic of this type of erosion is the badly worn reinforcing steel and aggregate as shown in the same figure.

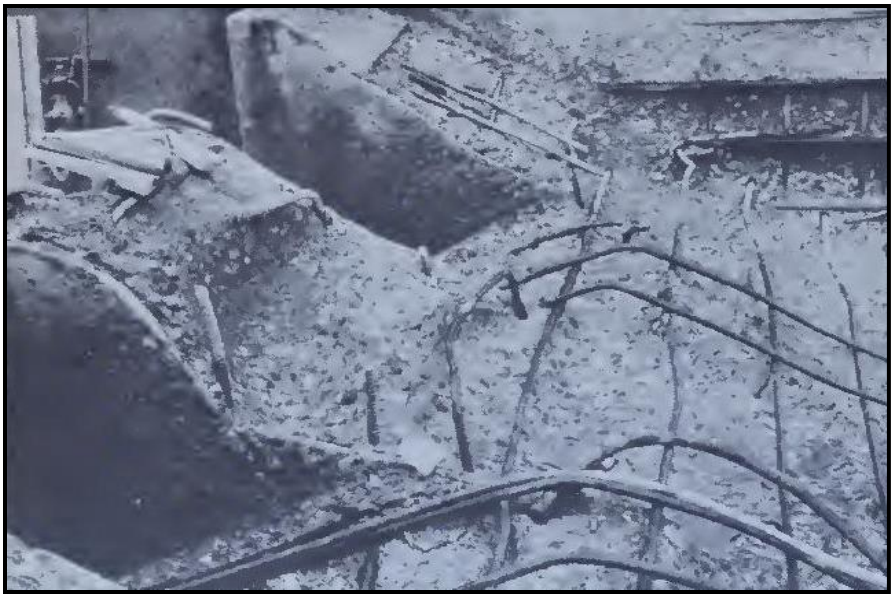

Figure 16: Abrasion erosion on concrete in the dentates, walls, and floors of the Yellotall Afterbay Dam Sluiceway stilling basin. The "ball mill" action of cobbles, gravels, and sand in turbulent water abraded the concrete, thus destroying the integrity of the structure.

Although most severe cases of abrasion damage occur in the areas just described, similar damage could be expected in diversion tunnels, canals, and pipelines carrying wastewater.

Use of concrete of increased strength and wear resistance offers some relief against the forces of erosion brought about by movement of abrasive material, abrasion and impact of traffic, sandblasting, and floating ice [20].

If major repair or rehabilitation is required, the state of the structure must be studied carefully and compared to the original conditions. The final outcome will determine the best method and repair materials which are going to be used. One example of such work was the repair of abrasion erosion damage to a spillway water diversion structure shown in Figure 17. In this case, it was ascertained after surveying the extent of damage, that the cause of this damage was abrasion erosion. After cleaning, it became apparent that the defective parts were much more widespread and serious than originally thought as seen in Figure 18. In this particular case, concrete was easily removed to significant depth over most of the spillway. Thorough investigation was conducted by obtaining and examining concrete cores from several defective parts. It was only possible then to determine the best treatment needed [21].

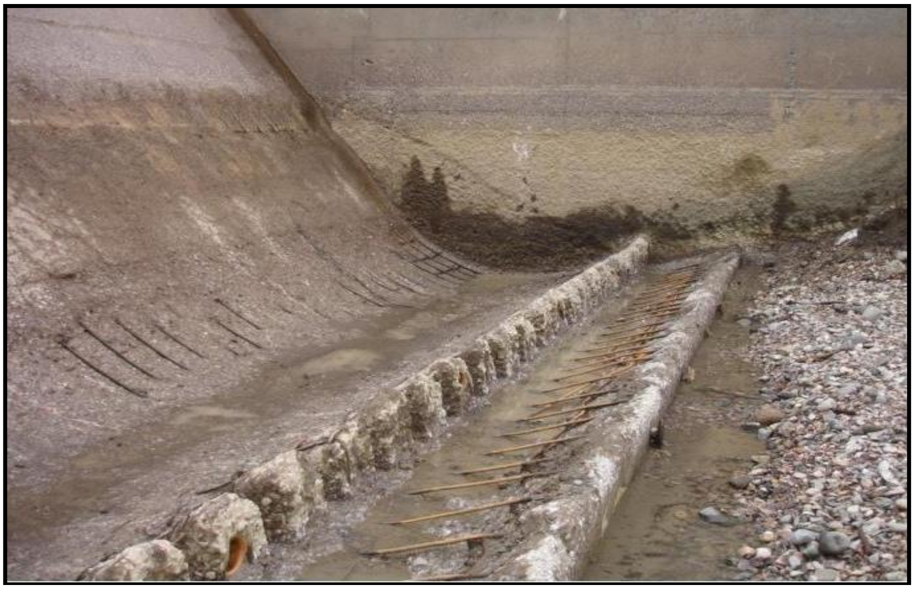

Figure 17: Abrasion erosion damage to the flip bucket and energy dissipators. The cause of the damage was not determined prior to starting repairs.

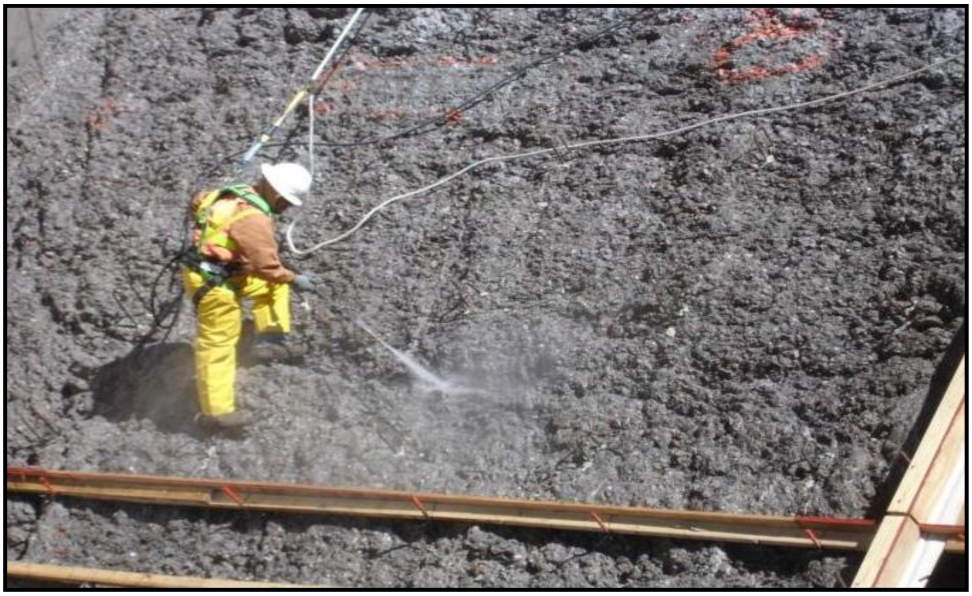

Figure 18: After beginning of concrete removal, the extent of damage became clear. Concrete was easily removed due to extensive deterioration.

One case study of similar damage is illustrated in Figure (19a) of another spillway chute damaged by abrasion. The damage was clearly visible on the surface of the spillway slab, side walls, pillars of the gates and the energy dissipation blocks. These parts were found damaged at several points which had been caused by abrasive processes. The location of the damaged part on the wall is indicated by the red circle as highlighted in Figure (19a). Similar defects on the spillway slab are shown in Figure (19b).

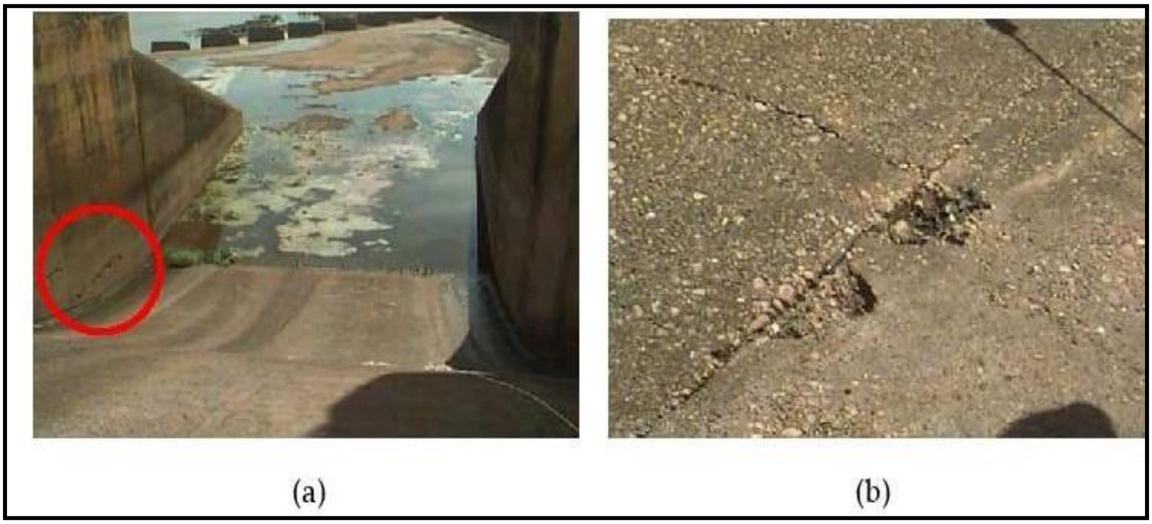

Figure19: (a) Dam Spillway deterioration by abrasion.
(b) Defect in the concrete structure caused by abrasive process.

Many alternative materials for repairing these defects could be suggested including, mortar with silica fume with optimum content of about 10% of cement mass replacing the fine aggregate, epoxy mortar, polymer mortar, concrete with steel fiber or high strength concrete with characteristics similar to concrete used in dam construction in a mix of 1:1.61:2.99:0.376 (cement:sand:gravel:water) with cement content of 426 kg/m^3 [22]. In another case study cited by the same reference, the repair of spillway chute of hydroelectric plant located in southern Brazil was done by adding polymeric materials to the concrete used to repair hydraulic surface degrades and damaged parts of the abrasive processes. The materials used was mixture of agglutinated low-density polyethylene (LDPE), crushed polyethylene terephthalate (PET) and rubber from useless tires. An overview of the dam spillway is shown in Figure (20a) Details of displacements in the surface caused by hydraulic abrasive processes are shown in Figure (20b).

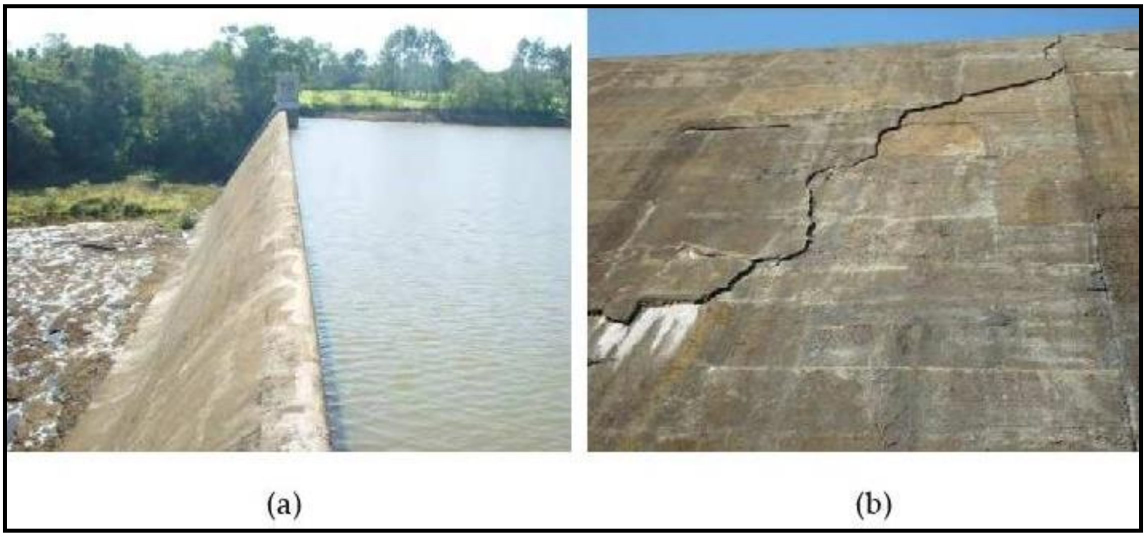

Figure 20: (a) Overview of the spillway of the dam. (b) Detail of the displacement of the surface caused by hydraulic abrasive processes.

After long service of old dams where roughened concrete surfaces have been developed already, the impact of abrasion erosion damage will be intensified and can be quite severe in large old dams and in the rivers of heavy loads of sediments.

6.2 Cavitation of Concrete

Cavitation occurs in concrete surfaces in spillway chutes, tunnels and stilling basins in dams and other hydraulic structures when their surfaces are subject to very high velocity flow. It is one of the most destructive mechanisms in such structures if it is allowed to start. Moreover, it is one to which concrete or any other construction material offers very little resistance regardless of its quality.

Cavitation is aggravated by aggregate pop outs, construction related offsets, and deposits of carbonates leaching product from concrete. Cavitation is triggered when high velocity flow is obstructed or met with abrupt change of surface alinement or surface roughness such as might occur on concrete surfaces misalignments because of poor formwork or inferior finishing of the concrete or due to deterioration of the concrete surface as the case may develop in older dams after long service in passing high floods.

The mechanism of cavitation develops when sub-atmospheric pressure zones of small fast bubbles are formed in contact with concrete or metal surface as very high velocity flow passes over them while the temperature remains constant. The bubbles instantaneous collapse results in concentrating tremendous amount of energy. The impact of the collapse has been estimated in some cases to produce pressures as high as 100,000 pounds per square inch. Repetition of these high energy blows eventually forms the pits or holes known as cavitation erosion. On spillways chutes and bottom outlets, cavitation may occur at clear water velocities of between 12 to 15m/s which may result in severe damage and can reach dangerous limits especially when velocities exceed 25m/s. Therefore, protection is needed at these velocities. Cavitation can be prevented by decreasing the flow velocity or by increasing the boundary pressure.

With velocities greater than 20 to 30m/s, the tolerances of surface finish required to avoid cavitation are too severe and the cost of cavitation resistant materials is prohibitive. For these reasons, it becomes usual to protect the spillway surface from cavitation erosion by introducing air next to the spillway surface using aeration devices located on the spillway bottom and sometimes on the walls [23]. Such aerators were added as a remedial measure to stop cavitation in the Glen Canyon tunnel spillways as will be described hereunder as a study case. In recent developments stepped spillway chutes are designed to increase turbulence in the flow and prevent sub- atmospheric pressures formation. Cavitation may occur on horizontal or sloping surfaces over which water flows or on vertical surfaces past which water flows. Figure 21 is an illustration of cavitation erosion on surfaces of adjacent stilling basin dentates [20].

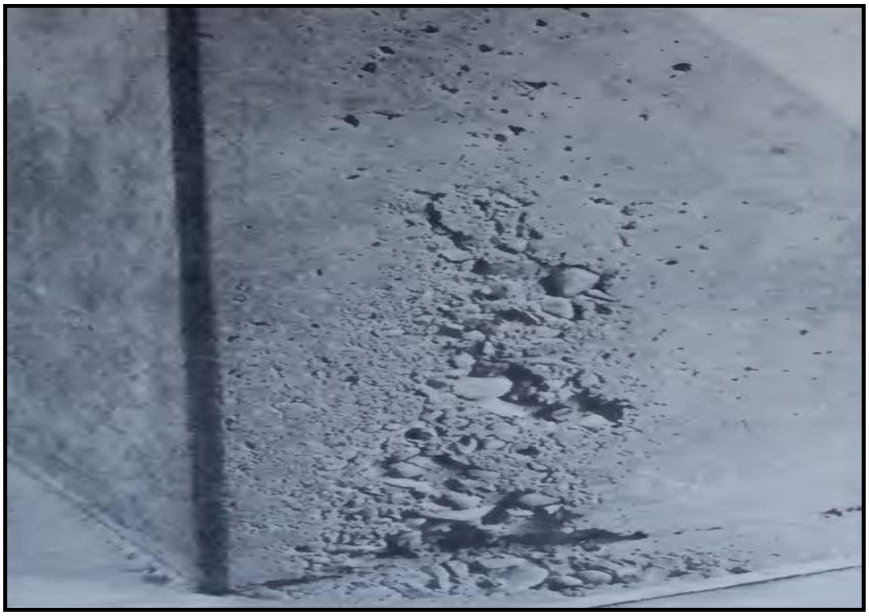

Figure 21: Cavitation erosion of concrete on a dentate in the Yellowtail Afterbay Dam spillway stilling basin. Fast-moving water during a flood flow, caused a pressure phenomenon at the concrete surface which triggered cavitation damage shown here.

Examples of major spillway damage by cavitation include Hoover Dam (USA), Aldea-Davilla Dam (Portugal), Yellowtail Dam (USA), Tarbela Dam (Pakistan), Karun Dam (Iran), Glen Canyon Dam (USA). In most cases, spillway aeration devices were installed as part of the remedial measures.

6.3 Glen Canyon Dam Case

Dramatic cavitation and subsequent erosion of the two emergency spillways at Glen Canyon Dam in 1983-84 required reconstruction of the spillways and installation of “air slots” to aerate the spillway flows. Glen Canyon Dam is a concrete arch-gravity dam on the Colorado River in Northern Arizona, United States. The 216m high dam was built by the U.S. Bureau of Reclamation from 1956 to 1966 and forms one of the largest man-made reservoirs in the U.S. with a capacity of 3.3 x $10^{9}m^{3}$. A 950MW power plant is located at the toe of the dam. An outlet, with four hollow jet valves, having a total capacity of 425m^3/s is located on the left abutment, refer to Figure 22.

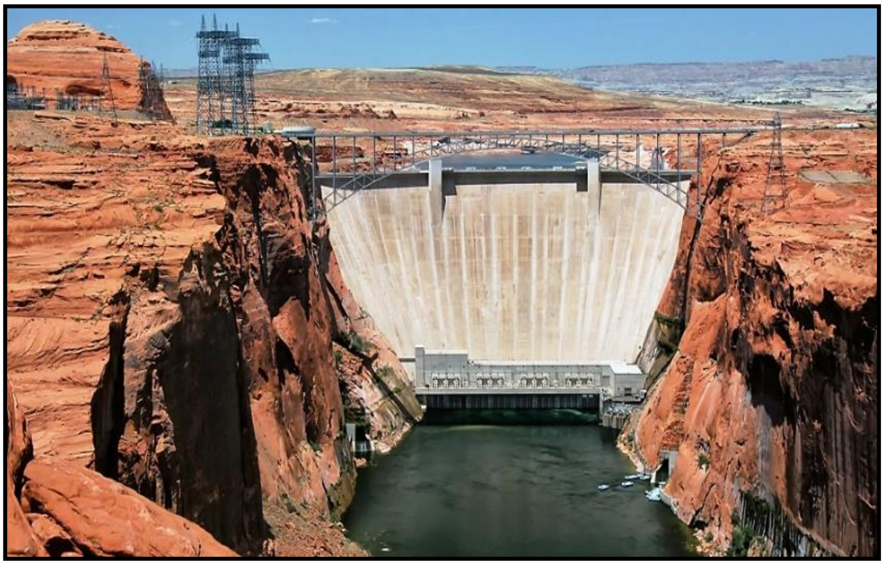

Figure 22: Downstream view of Glen Canyon Dam showing the Power Station and outlets of the two tunnel spillways.

The dam has two open-channel flow tunnel spillways, one on each abutment with the shape as shown in Figure 23. Each spillway is 12.5m in diameter and has a maximum capacity of 2900m^3/s. Flow of each spillway is controlled by radial gates which then passes into a 55° tunnel, a vertical bend (elbow), and 305m of horizontal tunnel to a flip bucket. Both spillways were operated for extended periods in 1980.

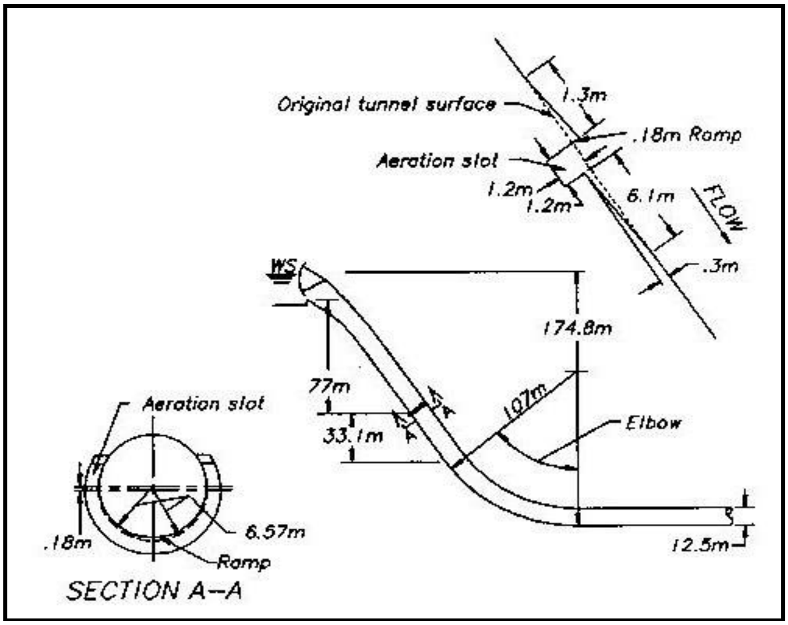

Figure 23: Glen Canyon, tunnel Spillways-aerator slots. (Upper Right-hand Corner).

In 1981, an inspection revealed that deposits from cracks in the lining had initiated cavitation damage at several locations in the left spillway. Little damage had occurred in the right hand side spillway which was probably due to the shorter operating time. Following this inspection, surveys and studies were performed to document the damage and to identify the scope of work required for repair and to prevent future occurrence. It was recommended the damage be repaired and aerators be installed near station 6+86.0. These modifications were planned to begin in 1984, but high flows in the Colorado River occurring in the spring of 1983 had to be passed through the spillway.

At the end of the flood, extensive damage had taken place in both tunnels as shown in Figure 24. Even though damage was extensive, it only excavated a hole of a depth of about 11 meters. At this point, the eroded cavity was evidently large enough to dissipate the energy of the high velocity water.

In the elbow portion of the tunnels, the depth of the damage as noted in Figure 25 was on the same order as the depth of the flow.

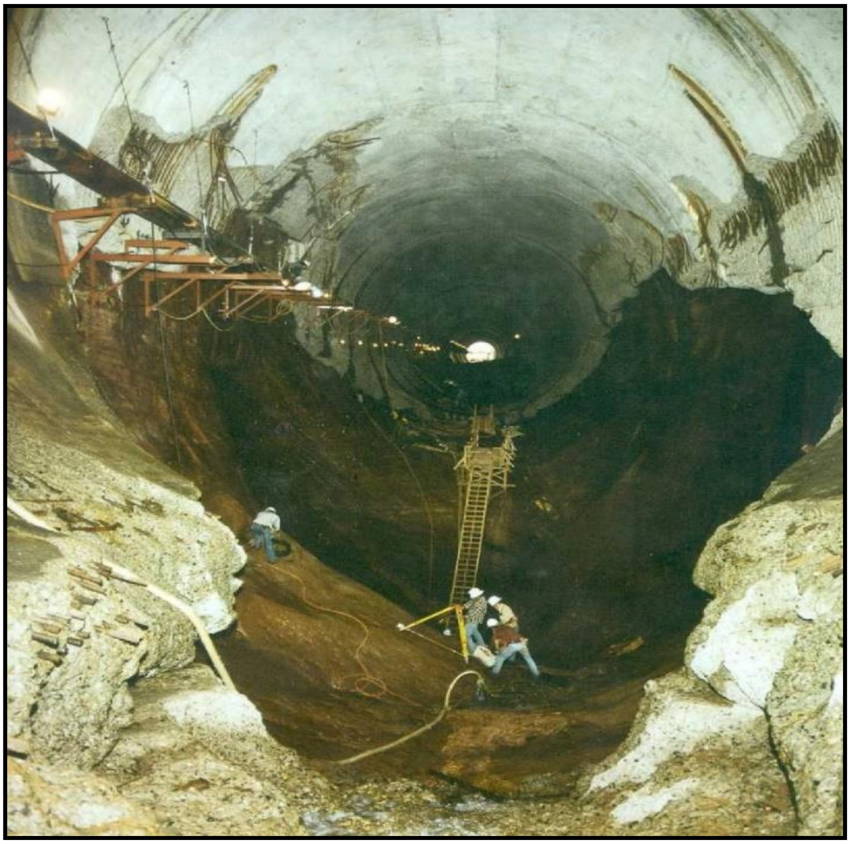

Figure 24: Glen Canyon Left Spillway Tunnel, September 1983. The "big hole" in the spillway invert was 11 meters deep.

The decision taken for repair was to construct aerators to inject air in the inclined portions of the spillways to stop future cavitation damage. After thorough study and modelling and design work, the aerators consisted of a ramp, a groove or slot, a downstream offset, and a transition back to the original tunnel diameter as shown on the earlier Figure 23 upper right corner. The ramp is 1295mm long and 180mm high at the centerline of the invert. The ramp feathers out to zero height at the springline. The groove is 1200 x 1200mm. The downstream edge of the slot is offset 305mm from the original tunnel diameter. The length of the transition to the original tunnel diameter is 6.1m. The end of the ramp is at station 6+86.5 which is 96.2m below the maximum reservoir elevation. The tunnels were ready for service after finishing construction on 10 October 1984.

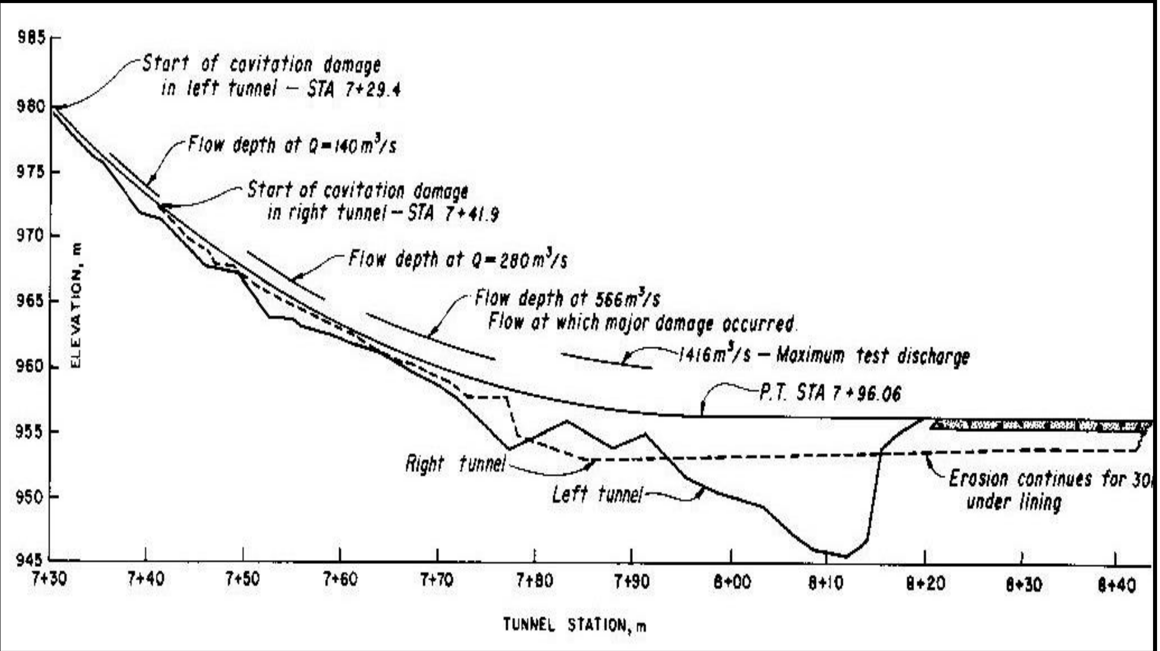

Figure 25: Glen Canyon Dam, tunnel Spillways- damage profiles.

In August 1984, the left tunnel was tested to verify the operation of the aerator. Pressure measurements were taken in the invert to compare with model studies and air velocities were measured in the air groove to estimate the airflow quantity. Flow rates up to 1416 cubic meter per second were passed through the spillway. This discharge was 40% greater than had previously passed through the spillways. In addition, a discharge of 566 cubic meter per second was maintained for 48 hours. This flow rate and duration had caused damage to the spillway during the 1980-81 flows. Evidence of large damage could not be found following the tests even though some eroded areas, at the end of the elbow, had been left unrepaired.

At Glen Canyon Dam, in the section downstream of the elbow, only damaged sections deeper than 19mm were repaired. Depending upon the extent of the damage one of two methods of repair was used. For small areas, the damaged surface was ground to a 1:20 (vertical to horizontal) slope in the direction of flow and to a 1:5 slope perpendicular to the direction of flow. For large damaged areas, the concrete was removed down to the first mat of reinforcing steel and then replaced with a concrete patch. Bolts and expansion anchors, which could not be removed, were cut off above the surface and ground down to be flush with the surface. Cutting off the bolts below the surface and plugging the hole, with hydrated cement or "Burk Water Plug" compound, was limited to bolts or anchors in which the concrete around them had spalled [24], [25].

A similar cavitation problem in a very large dam had set the precedence of damage caused by cavitation before the Glen Canyon Dam case; this was the case of Hoover Dam.

6.4 Hoover Dam (USA)

This Dam is 226.4m high concrete gravity arch dam on Colorado River which impounds Lake Mead, the largest reservoir in the United States with volume of $35.2x10^9m^3$ when it is full. The hydropower station of this dam had undergone upgrading from 1986 to 1993, making the total gross power rating for the plant 2080 megawatts. The average power generated was 4.2TWh/year for the period 1947-2008. In 2015, the dam generated 3.6TW/h. Plan view of the dam is shown in Figure 26; it illustrates the details of the dam and the appurtenant structures including the two gated spillways intakes, followed by the spillways tunnels; one on Arizona side and the other on Nevada side of the river. Construction of the dam and appurtenances was completed in 1936, but the two spillways (the Arizona and the Nevada tunnel spillways) were not used until 1941. At that time, a relatively small average discharge of about 384 cubic meter per second was passed through each of the tunnels for about four months.

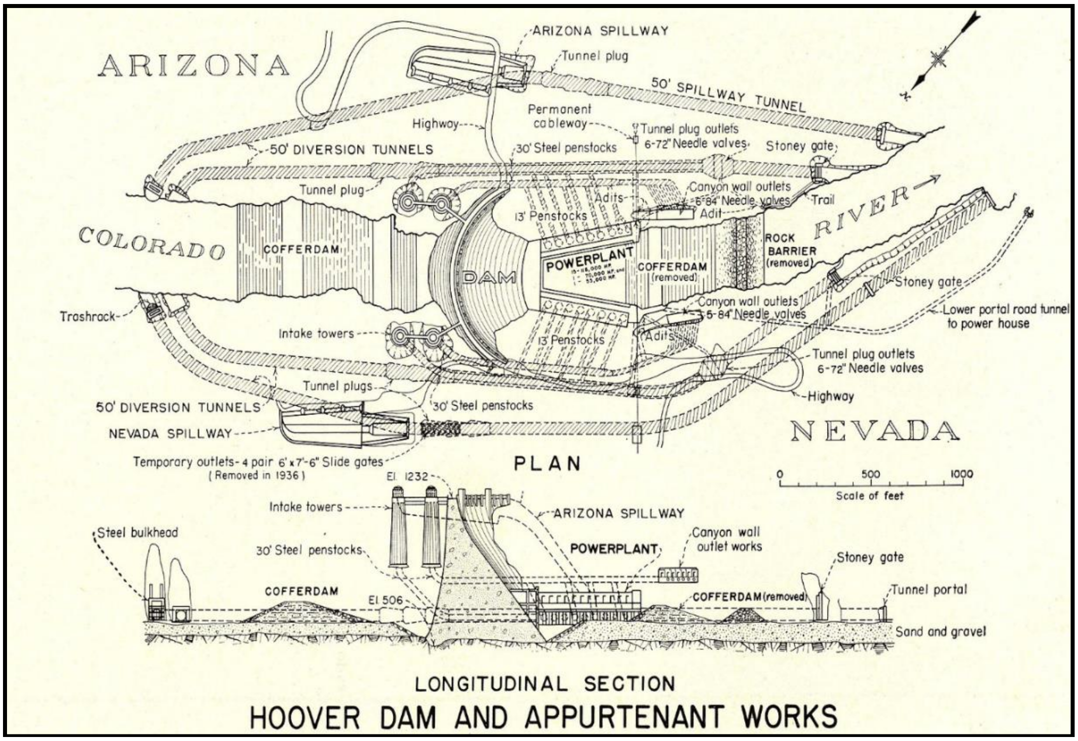

Figure 26: Hoover Dam Plan and Appurtenant Structures.

During testing of the two spillways in 1941, the drum gates were raised, and the reservoir continued to fill reaching level 1220.45 feet by July 30; within 11.55 feet of the dam crest. On the 6th August, the drum gates were gradually lowered and several months of spillways testing ensued, which continued through early October. View of water discharging from lake Mead over the four drum gates of the Nevada spillway during the spillway tests is given in Figure 27.

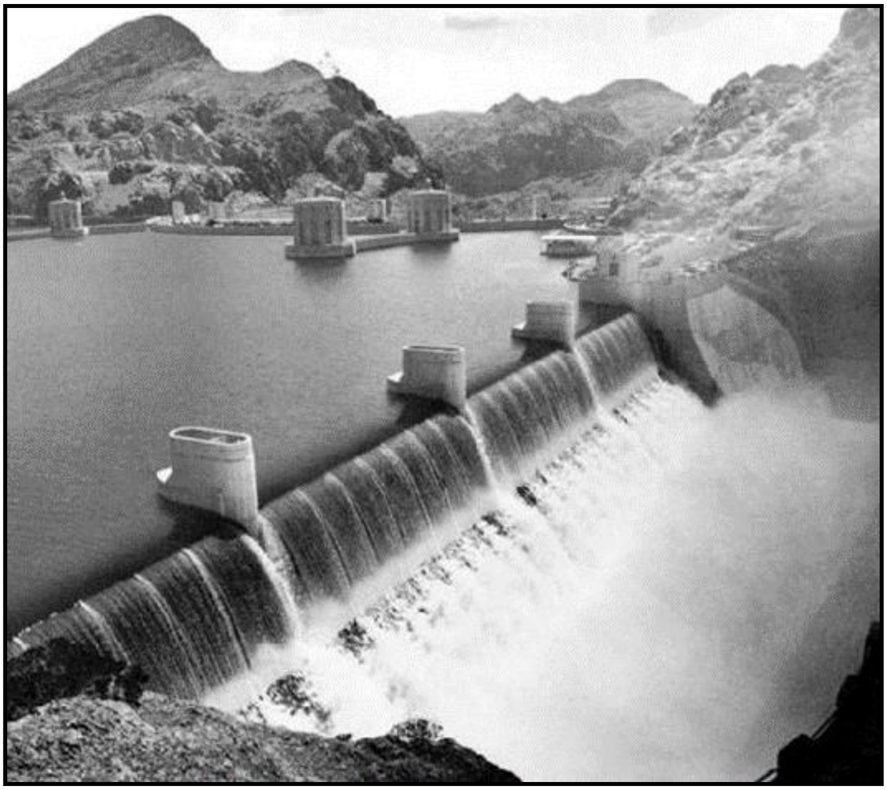

Figure 27: Water from Lake Mead discharging over the four drum gates of the Nevada spillway during spillway tests in August and October 1941.

Relatively modest flows, never exceeding 368 cubic meter per second were passed through both spillways for four months. Even with these modest flows, velocities at the elbows reached 175fps (53.3m/s) and cavitation damage ensued on both spillways. The cavitation was most severe on the Arizona spillway elbow, where a hole 112ft long, 35ft wide, and up to 36ft deep was eroded into the high strength reinforced concrete on this spillway; this is illustrated in Figure 28 [26].

The original volume of the cavity was 1069.6 cubic yards (818m^3). Repair work was started almost immediately, but because it was believed that ordinary concrete was not suitable it was decided to utilize the (Prepack and Intrusion) process of concrete repair developed by Durite of Chicago. After repair, the tunnel was polished smooth to help prevent any future erosion [27].

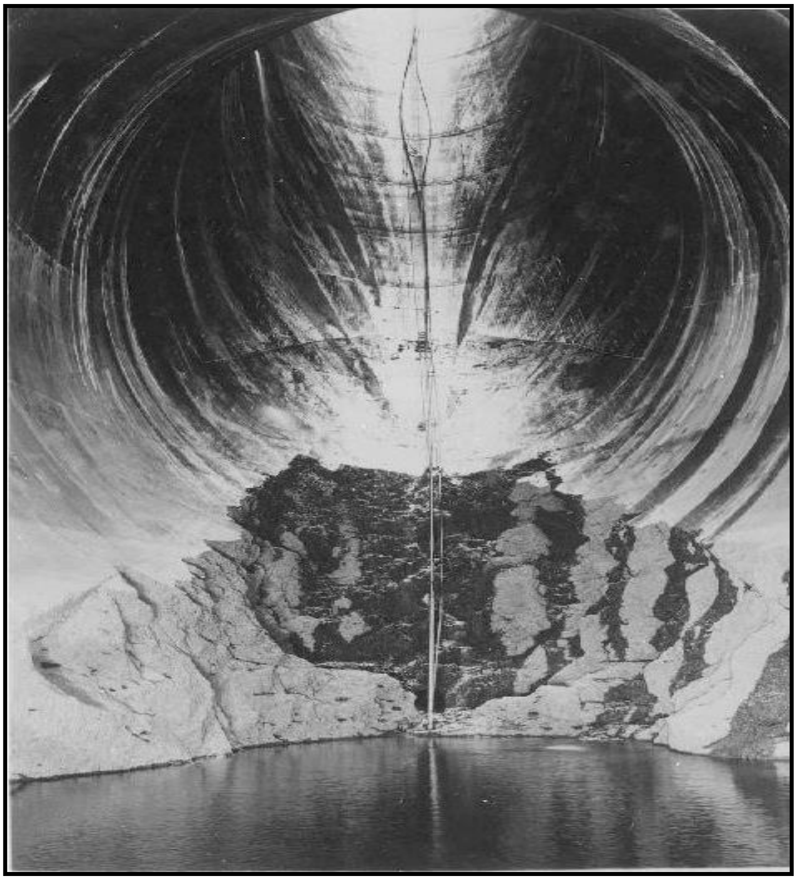

Figure 28: Cavitation damage observed in December 1941 in the elbow of the Arizona spillway after modest discharges passed through the spillway between August and October 1941.

It was judge that the most probable reason of this damage was being an abrupt misalignment in the tunnel invert that deflected the high velocity flow causing sub-atmospheric pressures and the formation of vapor pockets. Subsequent implosion of these vapor pockets further downstream caused cavitation damage which destroyed the integrity of the concrete lining and eventually led to the massive erosional damage.

Hydraulic model testing to design a good mean for injecting air into the flow as a corrective measure which followed, gave negative results, but the tunnel spillways, however, were not used again until the 1983 floods. Comparatively low flows (less than 283m^3/s) through each spillway were discharged for several hundred hours. Minor cavitation damage occurred, more in the Nevada than the Arizona tunnel. Since it was known that this type of damage is cumulative, it was decided that efforts on installing new aeration devices in both tunnels should be resumed [28].

From 1941, the year of the first model testing, till 1983 more had been learned about the nature of cavitation problems which allowed designing, testing, and installing more successful aeration devices in the two tunnel spillways. The effectiveness of aeration has been thoroughly demonstrated, however, the aeration scheme must be well designed to assure that it is effective through the entire range of operations. After the 1983 floods, reclamation undertook a comprehensive program to retrofit their high dams to alleviate cavitation, using aeration slots. These slots were added to the Hoover Dam spillways in 1985-1986 [29].

In summary, cavitation damage could occur in newly built dams, if the hydraulic conditions as described are favorable. The chances of the same phenomenon to develop in old dams are much more. In these dams cracking or displacements of concrete surfaces and misalignments which are subject to the high velocity flow, in addition to erosion by high floods, settlement cause by earthquakes are all good reasons to develop cavitation in these dams. In such cases, therefore, cavitation damage are to be looked for during routine inspections and prompt treatment are carried out, which normally are not cheap jobs.

7. Conclusion

From the proceedings it is clear that aging concrete dams present serious problems of safety hazard on downstream communities and can cause fatalities and material damage in case of failure. Major rehabilitation work in the present economic environment may require large investment in view of soaring costs. In addition to diminishing benefits and soaring costs of repair and upgrading. As conclusions from this presentation the following may be drawn.

1. Concrete dams design and construction methods have evolved over more than 120 years. At the present such dams may present safety risks by the fact that data and methods of analysis used at those days have mostly become obsolete. The decision to repair old concrete dams must be based on reviewing their general safety as to stability and integrity together with evaluating all the damages sustained during these years.
2. Aging processes of concrete dams result from both external and internal factors. These factors include temperature variations, freeze and thaw, settlement, earthquake vibration, abrasion and cavitation by high velocity flow, blocked drains, chemical reactions between concrete constituent’s material and water as in alkali aggregate reaction and sulfate attacks, and corrosion of metal parts. While some of these factors are unavoidable by their nature, the effects of others can be reduced by good design and controlled construction.
3. The aging of these dams is a reactive process between the elements of nature and the dam structure and again reactive it is reactive process between its constituents which renders these dams’ unsafe sooner or later.
4. The relatively long history of concrete dams’ construction has given lot of opportunities to encounter various types of damages. Using study cases of many of such occurrences has accumulated rich experience in the causes and effects and this allows the construction of better dams. To say that the life of any dam can be elongated indefinitely is false. It is the nature of all man-

made structures to decay with time and nothing can be made to stop this decay. Slowing down of this can be achieved by good design and by repairs and upgrading until such things become technically and economically not possible. After this the only alternative left is tearing down the dam and removing it.

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Dam Safety: Technical Problems of Aging Embankment Dams

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and Sven Knutsson[5]

Abstract

Embankment dams undergo aging process due to the impact of different factors which can be attributed to geology of the site, design of the dam, materials selection and procedures followed in constructions. In the foundation the presence of faults or shearing planes, karst, compressible clayey material, soluble rock, and soft rock may establish conditions leading to high total settlement or differential settlements of the dam and its cracking. Deficient and deteriorating seepage control measures such as grout curtains or diaphragms enhance seepage flow leading to internal erosion and piping which endanger dams' stability. Improper filling materials used such as dispersive clays and gap graded granular material show their bad influence after long time by creating conditions inducive to internal erosion and piping. Use of improperly designed and placed filter zones and drainage blankets can end in clogging of such filters and drainage blankets leading to the rise of the phreatic surface level and increasing uplift causing again conditions of internal erosion and piping and undermine stability. This work attempts to give an overview of these conditions and cite many case studies of rehabilitation works carried out in dams after long years of service. The conclusion reached is that rehabilitation works if done early when problems are discovered play well to elongate the service life of dams, but normally they require large investments. Sooner or later owners of such old dams will come to realize that more rehabilitation works, neither technically nor economically, are feasible and that more of such works are not possible. In which case they will come think seriously of decommission such expired dams.

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1. General

Statistics show that a new earth fill dam is most likely to fail during the first five years of service. This may be quite understandable as the dam is tried for the first time during this period especially through first filling. During this period undiscovered anomalies in the foundation or flaws in design or construction will show their effects and take their toll while settlement of the fill material is highest with the possible cracks development. Deterioration in the dam component, such as filters, drainage facilities and protection works, in addition to slow detrimental changes in the foundation such as compression, dissolution of rock and soil materials, or grout curtain failure will take longer time and might take between ten to forty years depending on how much attention and monitoring have been done and how much maintenance works have been applied during these periods. One study based on data compiled from 900 dam failures of all types and published in 2007 showed that 66% of these failures belong to earth dams. The same study indicated as shown in Table 1 the age of the 593 failed dams at the time of failure.

Table 1: Age of failed earth dams at time of failure [33].

Age range of cases	Number	Percentage (%)
0-1	85	14.3
1-5	96	16.2
5-10	36	6.1
10-20	62	10.5
20-40	58	9.8
40-60	31	5.2
60-80	16	2.7
80-100	7	1.2
100-150	10	1.7
>150	6	1.0
Unknown	186	31.3
Sum	593	100.0

The study further indicated that dams constructed during two periods, 1890-1939 and 1950-1979 appear to have suffered the highest rate of failure as shown in Table 2 [1]. The first period (1890-1939) was a period of trial and error and learning from mistakes when design tools, input data and construction methods and materials were not available in the way they are today, while the second period (1950-1979) reflects the boom in dam construction all over the world, where even the small probability of failure can result in large number of such failures [1].

One limitation of the statistical approach in predicting dam failures is the fact that site conditions of all studied dams were not the same in terms of geology, hydrology, or seismicity and even in terms of design procedures used. This entails careful scrutiny of all these conditions case by case in judging the possible impacts of age before including them in such a study.

The survival of many old dams suggests that their engineering has been amply conservative, or that their worst trials are yet to come. However, the level of conservatism in the design of an individual dam cannot be determined without a verified comparator. The ultimate measures are the actual behavior under extreme conditions [2].

Table 2: Construction time of the failed earth dams.

Construction year range	Case number	Percentage (%)
Before 1800	**8**	**1.3**
1800-1849	**11**	**1.9**
1850-1859	**8**	**1.3**
1860-1869	**14**	**2.4**
1870-1879	**5**	**0.8**
1880-1889	**21**	**3.5**
1890- 1900	**32**	**5.4**
1900-1909	**38**	**6.4**
1910-1919	**50**	**8.4**
1920-1929	**41**	**6.9**
1930-1939	**31**	**5.2**
1940-1949	**22**	**3.7**
1950-1959	**38**	**6.4**
1960-1969	**53**	**8.9**
1970-1979	**36**	**6.1**
1980-1989	**9**	**1.5**
After 1990	**2**	**0.3**
Unknown	**174**	**29.6**
Sum	**593**	**100**

2. Nature and causes of aging in embankment dams

The aging of dams, whether concrete, masonry or earth fill dams is a natural phenomenon common to all man-made structures, and it is the outcome of an active and never stopping processes of deterioration and decay. As time passes, this degradation continues in dams in a process similar to the aging of human beings, for this is a result of dynamic activities of interactions within the dam components which have inherent weaknesses that are seeded by design or construction procedures in addition to reactions towards the external forces acting on the dam.

Degradation processes work in dam foundation and in dam body as well. If dam foundation is within active fault zone, this may cause dam failure in case of a ground movement, whether this is caused by seismic action or loss of shearing strength. Moreover, if the materials of these foundation are highly compressible then large settlement and cracking or loss of freeboard of the dam will follow. Similarly, if the foundation is formed of soluble materials, then the leaching effect of water will erode such materials causing loss of strength and dam collapse. Similarly, if seepage control measures in these foundations, such as cutoffs, grout curtains and relief wells, are not successful then extra uplift and high exit gradient will cause piping and failure.

Degradation within the embankment may happen due a wide range of causes which have negative impacts on its stability if they are allowed to continue for appreciable length of time.

From these causes,

i. High pore-water pressure caused by high water content of the placed fill material and over-compaction which brings with it hydraulic fracturing and gradual piping and failure.
ii. High pore pressure means also reduction of shearing strength and reduced factor of safety against sliding in case of any increase of loading such as in un expected earthquake.
iii. High pore pressure resulting from clogged filters and drainage zones or blankets brings saturation of the downstream and rise of the phreatic surface above the toe of the dam. This clogging is the outcome of either improper design of these filters or due to bad selection of materials. The rise of the phreatic surfaces will cause piping and sloughing with the progress of time.
iv. Improper selection of the embankment core materials by using clays of dispersive or expansive nature which brings dispersion and hydraulic fracturing problems with them, or for the shells, the use of gap graded materials which brings with it suffusion. In such cases piping and internal erosion become highly probable.

All these matters should mean much worry and concern about the safety of the dam, not only to the designers but to the caretakers also. These concerns extend throughout the entire life of the dam until its safe abandonment or demolition.

Design procedures may mitigate effects of aging. Vigilance during construction may correct conditions contributing to aging. Monitoring during operation can identify aging processes which could impact on dam safety. The aging of dams constructed of earth and rockfill materials as discussed here is mainly, however, due to time-related changes in the properties of the materials of which the structure and its foundation are composed.

3. Foundation problems in aging embankment dams

Historical failures show that very high percentage of such failures are caused by foundation defects. These defects have gone unnoticed during investigation and design phases or they were ignored because of the inherent uncertainty associated with their nature. The impacts of these anomalies appear however after many years of service. The failure of Baldwin Hills Dam may serve as a good example.

3.1 The Baldwin Hills Reservoir Dam Case

This dam was constructed in 1951 to impound water into its reservoir to provide water to the south and southwest portions of the city of Los Angeles in California. The reservoir was confined on three sides by compacted earth dikes and the Baldwin Hills Dam on the northern fourth side. The Baldwin Hills Dam had a height of 232 feet and stretched a total of 650 feet in length. Its failure occurred in 1963 after 12 years of service. The designers of the Baldwin Hills reservoir and dam recognized since the beginning the difficulties associated with the site on which they planned to build the structure. But they underestimated the consequences of leakage of the reservoir if such leakage would occur.

At the location of the dam and reservoir, the immediate subsurface was comprised of loose, sandy soil followed by large block-like schist formations. In an effort to prevent water inside the reservoir from contacting the soft erodible soil beneath it, the reservoir was lined with asphaltic lining and equipped with a complex underdrain system. A typical section of the drainage system between the water in the reservoir and the dam embankment is shown in Figure 1. Moreover, the designers acknowledged the Inglewood Fault System that underlies the Baldwin Hills area. They assumed, however, and without any logical reason, that any movement or subsidence that may occur as result of the fault system would not be large enough to compromise the integrity of the brittle reservoir lining [3].

On the morning of December 14, 1963, during a routine daily inspection, the reservoir’s caretaker noticed that water had begun draining from the pipes beneath the asphalt membrane lining in large quantities. Personnel on site engaged the outlet works designed to lower the reservoir in emergency situations and alarm was sounded in the downstream area and 1600 persons were evacuated from their homes.

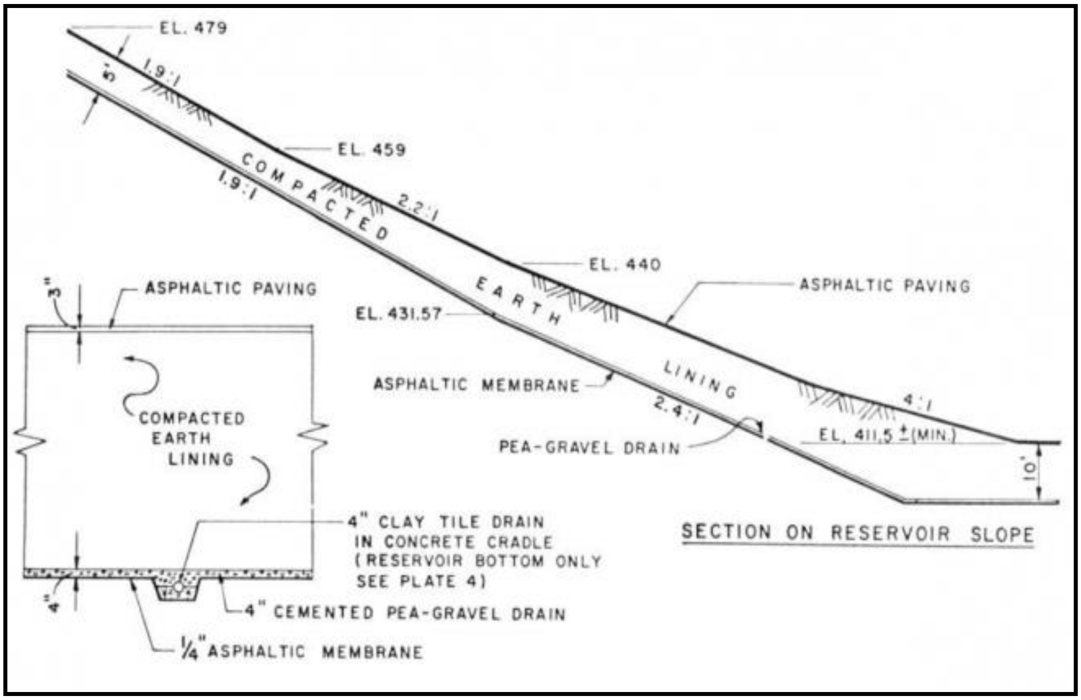


Figure1: Typical section of Baldwin Hills Reservoir lining.

Despite of all efforts, one section of the Baldwin Hills Dam collapsed after three hours releasing huge destructive wave on the town below. The total release of 950,000 m^3 of water resulted in only five deaths, thanks to the early sounding of the alarm and evacuation of the residence but the destruction of 277 homes could not be avoided. Vigorous rescue efforts averted a greater loss of life, refer to Figures 2 and 3.

As the reservoir completely drained it was revealed that the asphalt lining had cracked allowing water to penetrate and erode the soil beneath it. In the aftermath of this failure there was much speculation of the primary cause(s) of the crack during the investigation and law suits that followed the failure which are outlined in the following.

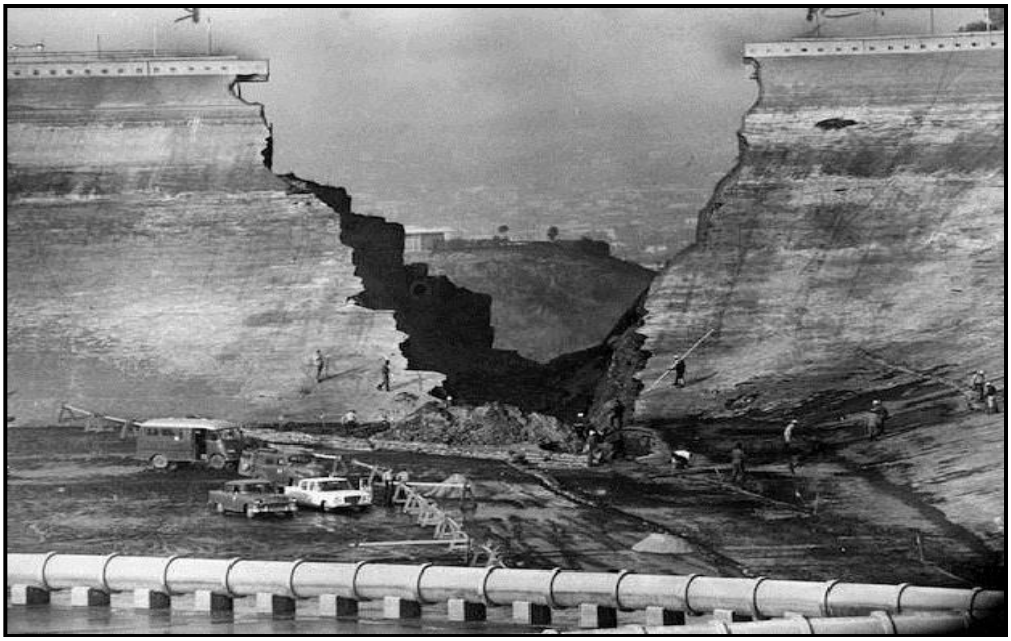


Figure 2: Baldwin Hills Reservoir Dam showing breach section in the aftermath of failure.

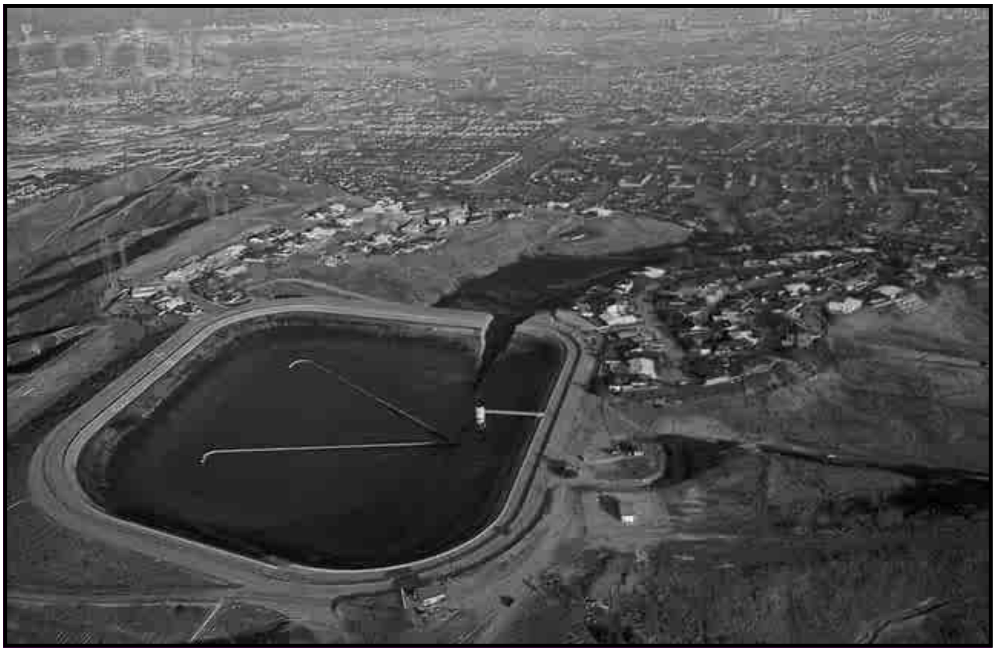


Figure 3: Aerial view of Baldwin Hill Dam Reservoir showing breaching section.

In its investigation, the State Engineering Board of Inquiry determined that the rupture of the reservoir lining which ultimately caused the collapse was result of slow earth movement concentrated at one of the fault lines. This movement was claimed to be caused by the tectonic movement manifested by the general subsidence in the Inglewood fault system area as visualized in Figure 4 [4].

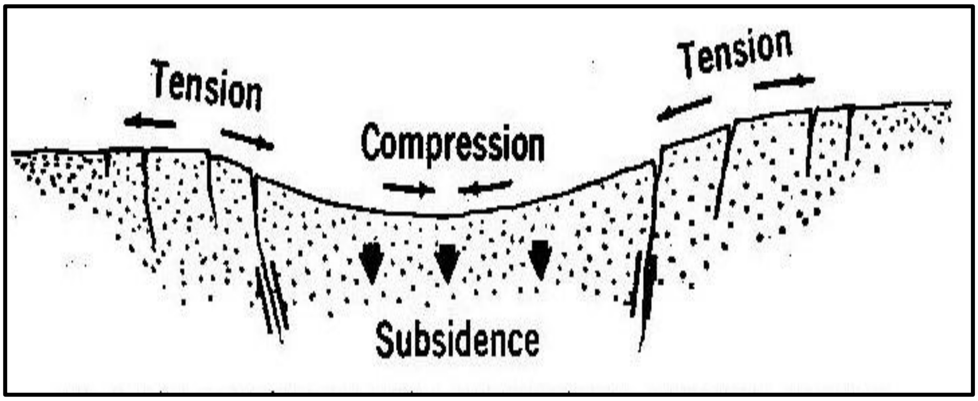


Figure 4: Diagrammatic representation of fault activation attributed to subsidence [4].

Two law suits filed in 1966 by the city and its insurers against the oil companies active in the Inglewood oil field at the time of dam failure charged that the oil operations had led to the events directly associated with the breaching. It was claimed that the injection of water under high pressure used in the extraction of oil may have weakened the uncemented rock, practically causing marked changes in the geologic fabric of the foundation and the earth-crack ground rupture was generally related to high pressure injection of fluid to the previously faulted and subsidence stressed seems. The fault activation leading to ground rupture appears to be a near surface manifestation of stress relief faulting triggered by the fluid injection, a mechanism identified as being responsible for the 1962-65 Denver earthquakes and for the generation of small earthquakes at Rangeley oil field in western Colorado [4]. According to Robert Jansen, a premiere consulting civil engineer, the foundation of the reservoir had been subjected to progressive horizontal stretching, concentrated at the steep fault planes in the soft rock as visualized in Figure 4. The foundation schist blocks under the reservoir literally tended to pull apart and drop down in a staircase, and then rebound. This action formed gaps between the blocks that became ready conduits for leakage once the lining was destroyed [5]. The rupture allowed water to escape into the loose partially consolidated sandy soil under the reservoir and the dam failed [5].

The case of Baldwin Hills Dam serves as good example of slow deterioration process in the underlying foundation which had taken twelve years to result in such sudden failure. In spite of the continuous inspection and monitoring of the reservoir and the dam the real hidden foundation defects escaped the attention and even baffled the investigators afterwards.

3.2 The Wolf Creek dam case

Piping in dam foundations, which can pause threats to dam safety, may result from geological features incorrectly evaluated or underestimated and lead to constant worry and costly repairs over the life of the dam. In this regard the Wolf Creek Dam Case serves as a classic and very illustrative example. This dam is multi-purpose dam on the Cumberland River in Kentucky, United States. It was built by U.S. Army Corps of Engineers to generate hydropower, control flooding; and allow year-round navigation on the Lower Cumberland River by releasing the navigation flow required, and it creates Lake Cumberland for recreation with storage volume of 7.5 Billion cubic meter of water. Construction of the dam began in 1941 but because of World War II and other factors, construction was not completed until 1951.

Wolf Creek Dam is 1748 m long and 79m high dam with a combined earthen and concrete structure. The concrete section of the dam consists of 37 gravity monoliths that make 548 meter of the dam's length across the old river channel. The spillway section contains ten 15.00m x11.00m tainter gates and six 1.20m x 1.80m low level sluice gates. The power intake section contains the penstocks that feed the six 45MW turbines making total installed capacity of 270MW. The embankment section extends 1200 meters from the end of the concrete gravity portion across the valley to the right abutment. This non-zoned embankment is composed of well-compacted, low plasticity clays, from the valley alluvium Figure 5.

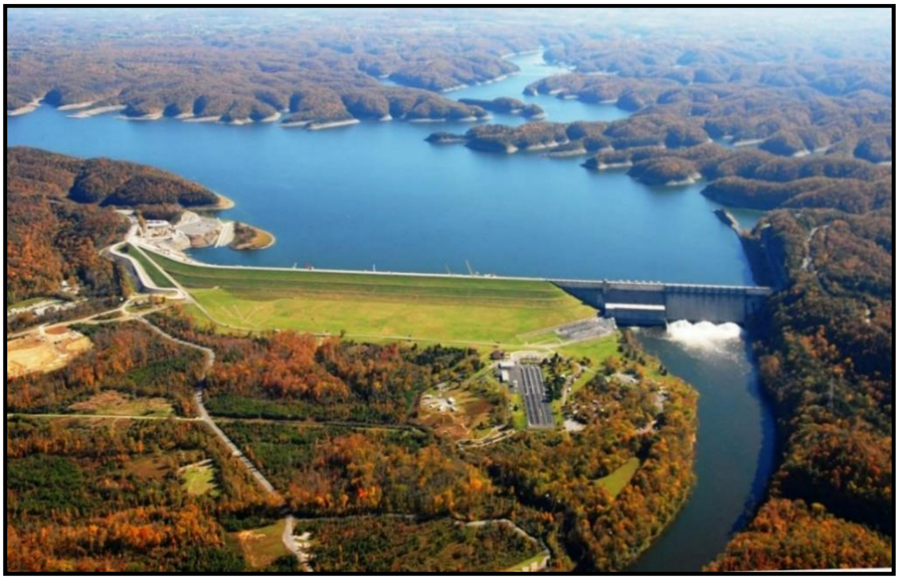


Figure 5: Overview of Wolfe Creek Dam, showing both concrete part and embankment.

The dam was built on heavily karstified limestone foundation. This foundation is composed of approximately 70% limestone and 30% shale units. In ascending order, the stratigraphic units that comprise the Highland Rim are the: Ordovician age Catheys, Leipers and Cumberland limestone, Devonian age Chattanooga shale and Mississippian age Fort Payne marine limestone. There is an unconformity between the Catheys and Leipers formations in which the Catheys was likely exposed. Sometime during the late Pliocene or early Pleistocene period uplift and erosion occurred. It is during this time that the karst features are believed to have developed. The Leiper's formation and upper five feet of the Cathey's formation are carved with solution features that impact the dam foundation. These two formations combined make up a total of 70m of limestone at the base of the dam. Overall, the bedrock is structurally intact without any faulting at the site. However, there exists jointing that trends both parallel and perpendicular to the dam axis. The systematic joint set that is normal to the dam axis is oriented approximately N 30º W and the limestone is known to be highly karstified. This fact is all the worse for its significant position under Monolith 37 and the concrete-embankment interface. A conjugate set of joints are oriented approximately N 75°E, and so being parallel to dam axis. This was the set utilized for the cutoff trench. The intensely karstified Leipers formation exhibits channels and caves developed along the near vertical joints and horizontal bedding planes. Geological section of the dam is shown in Figures 6 and Figure 7 [6].

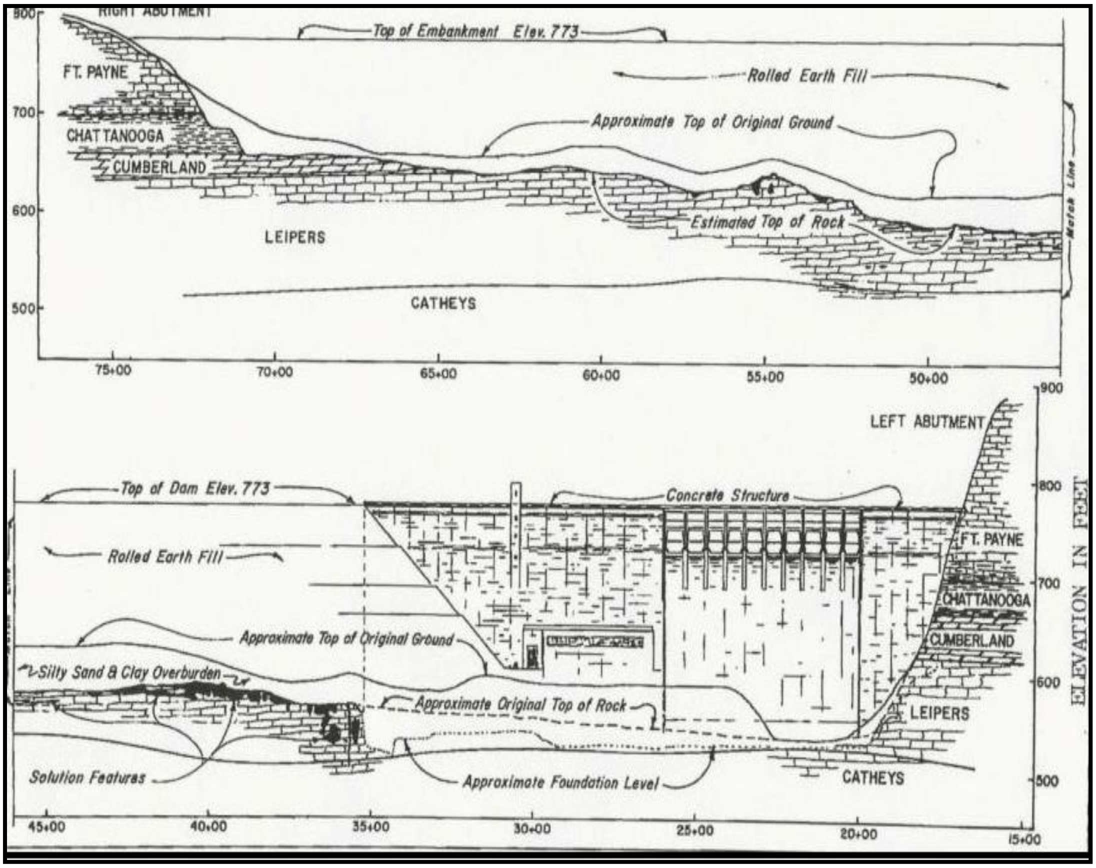


Figure 6: Geological section of Wolf Creek Dam.

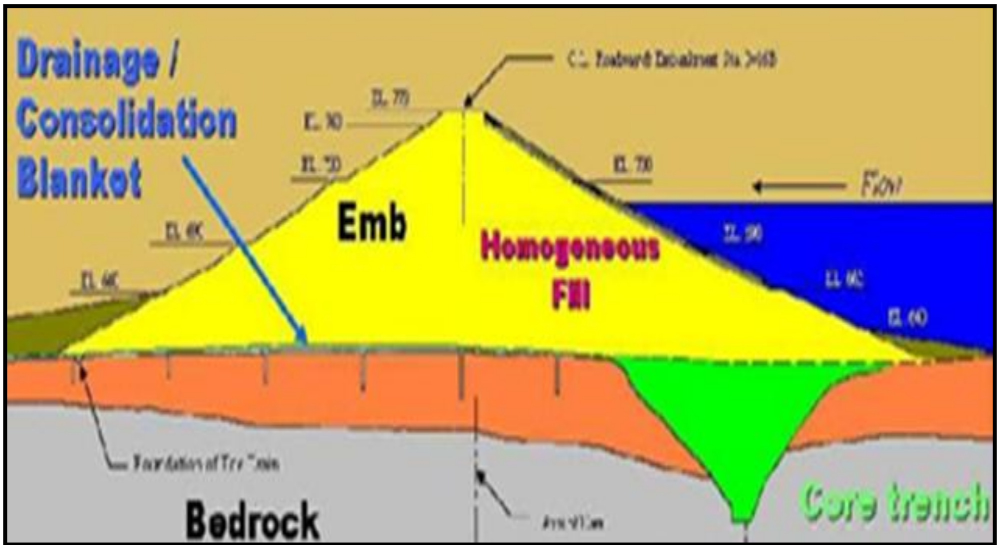


Figure 7: Embankment section of Wolf Creek Dam.

Large cavities and pronounced solution features were encountered during the excavation of the cutoff trench under Monolith 37 and the concrete embankment interface as illustrated by the photograph in Figure 8 [7].

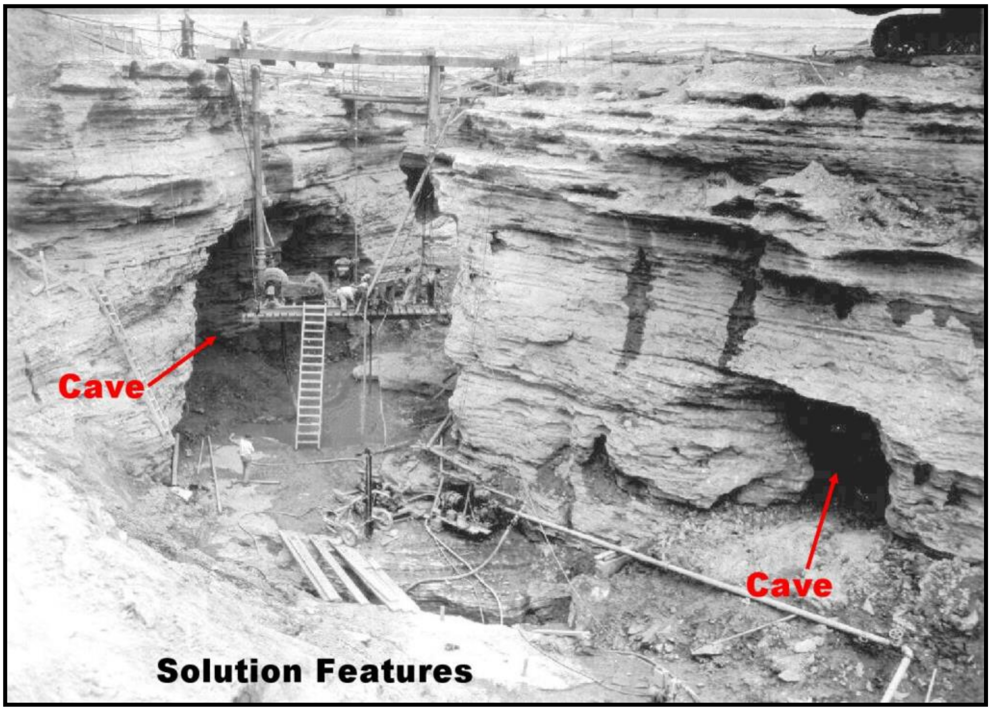


Figure 8: Cavities and Solution Features encountered in Cutoff Trench Excavation [7].

Seepage was first encountered during foundation construction and visible karst features were identified. The seepage problems were traced to the karst geology of the region which allows for the dissolution of limestone, and it is recognized to be in large part due to solution of the carbonate rocks lying beneath the embankment section. It is thought now that the foundation treatment techniques used were inadequate for this condition. Most of the alluvium was left in place and except for the cut-off trench, no treatment for the embankment foundation was performed, moreover, the cutoff trench design and construction were not properly done [7].

After 17 years of service, excess seepage problems within Wolf Creek Dam foundation began to appear. Beginning of 1968, signs of this seepage were discovered when two sinkholes appeared at the downstream toe of the dam and muddy water was observed in the dam's outflow channel. Solution channels caused by this process allow piping to occur which added to the rate of erosion in the foundation Figures 9 and 10. As concerns over this seepage increased one campaign of investigation was started which found solution channels in the limestone, and local piping.

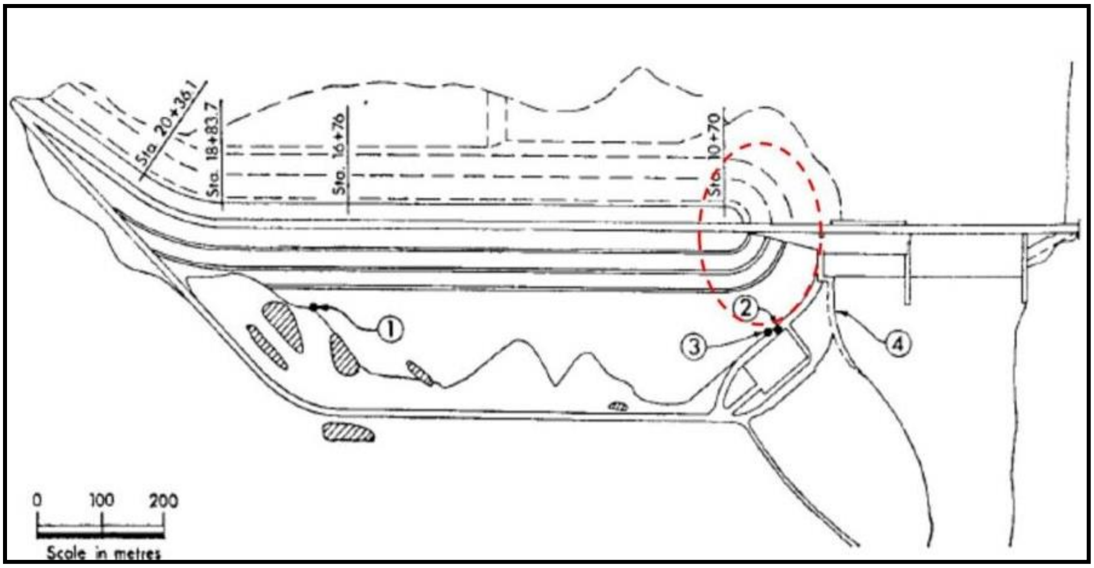


Figure 9: Sinkhole location: (1) sinkhole of 8/22/67. (2) Sinkhole of 3/13/1968. (3) Sinkhole of 4/22/1968. (4) Muddy water (Tailrace). (///) Wet areas, red dash outlines the "area around monolith 37".

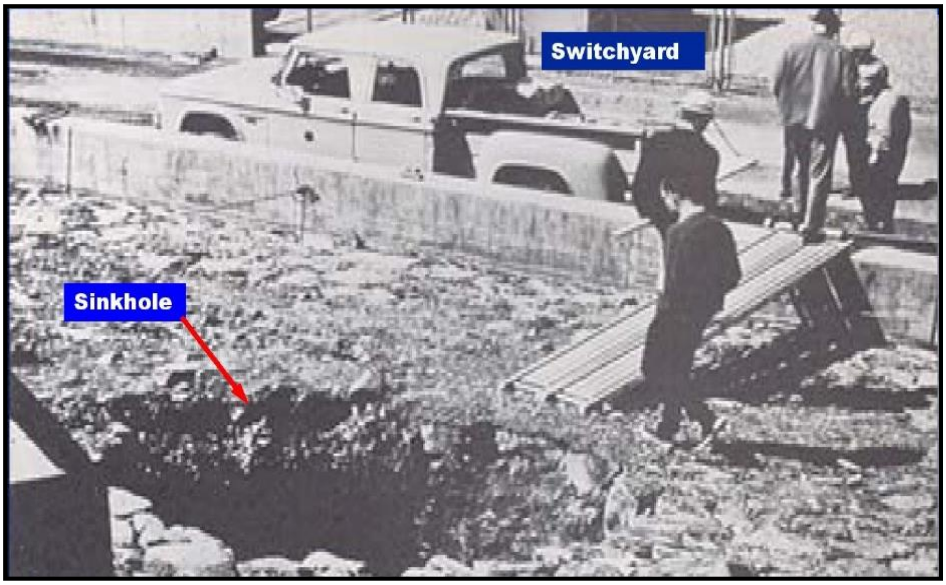


Figure 10: Sinkhole of 3/13/1968 at the Edge of Switchyard [7].

Emergency grouting over 1968-72 stopped the immediate piping, and it is generally accepted that this had saved the dam, but as recognized afterwards more was needed at the time. Following this, severe foundation seepage problem led to the necessity of a permanent solution at Wolf Creek Dam after the completion of the emergency grouting program completion in 1972. A board of consultants was convened whose adopted recommendations were to install pile type concrete walls diaphragm through the embankment of the dam and the area between the switchyard and the tailrace. This type of highly specialized work was not done by the USACE before which led to splitting the construction into two phases, with approximately half of the embankment wall and all of the switchyard wall in Phase I, and the remainder of the embankment wall diaphragm, which was the trickier part in Phase II. The walls were comprised of 26-inch steel cased tremie concreted caissons on 4.5 foot centers connected by concrete elements tremied through the excavated trench excavation in between. All excavation was conducted through a full head of bentonite stabilizing slurry. The wall reached a maximum depth of 278 feet, and the diaphragm wall construction was accomplished from 1975 through 1979, and site restoration completed in 1980.

Instrumentation of the dam behavior, after completion of this work was taken very seriously. The dam work was highly instrumented and carefully monitored. The instrumentation system included some 300 open pipe piezometers, pressure transducers, inclinometers, seismic instruments, and surface monumentation. Water temperatures were measured in the piezometers every two to three months. Geodetic monuments surveys were performed semiannually, unless specifically requested, and inclinometer readings were made at three month intervals. No problems were foreseen for the switchyard diaphragm wall. For the diaphragm of the embankment, although the concrete test cores indicated that the more vulnerable portion of the wall, the secondary elements, were probably fully closed, the possibility did exist that there could be windows in the structure. However, there was indications for under-seepage to have resumed below concrete structures founded in so-called sound rook, and constant attention to instrumentation and visual inspection were to be maintained. It was thought then that the most likely future problem would be at the right hand end of the wall following completion of Phase II, and the hydraulic gradient should be monitored closely to determine if future remedial action is to be necessary in this parts [8].

Monitoring of the concrete barrier revealed that the diaphragm wall had slowed but not stopped the seepage and erosion problems. The USACE admitted then that water seepage had found new paths under and around the wall, and even possibly through defects in the wall itself as erosion of solution features continued. The extent of the diaphragm completed in Phases I and Phase II is illustrated in Figure 11.

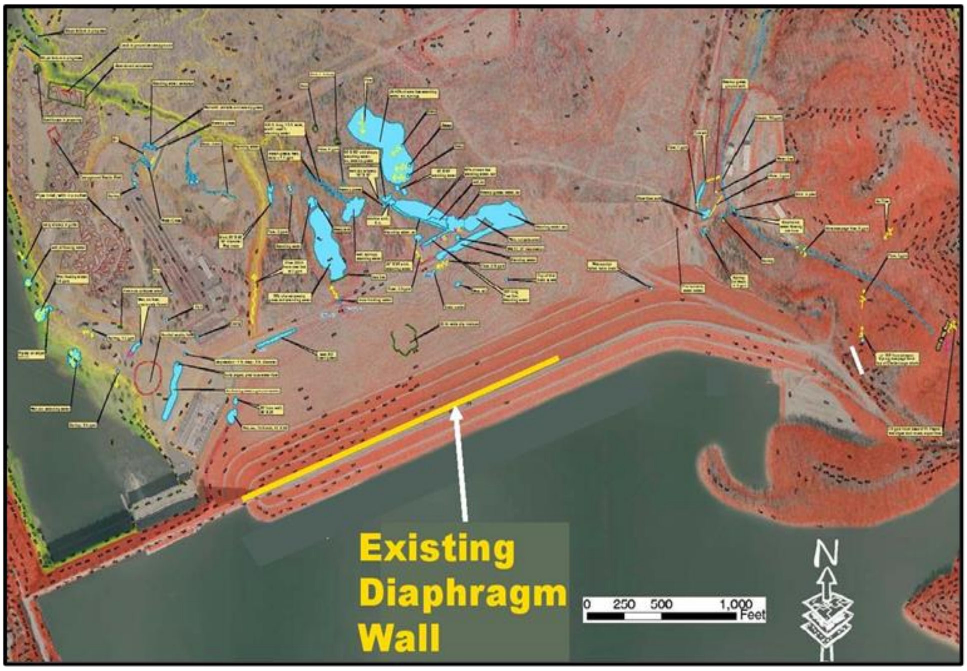


Figure 11: The first diaphragm wall of Wolf Creek Dam diaphragm wall as indicated in 2004 ([7] and [38]).

Further studies were undertaken over 2004-5, concurrently with USACE's development of its risk-informed safety program looking at all of its dams. Conclusions were that the diaphragm wall did not go deep enough into the bedrock, and it did not extend laterally far enough to intercept all major karst features [9]. Wet areas continued downstream of the dam and in 2006 an independent assessment by the Dam Safety Action Classification (DSAC) Peer Review Panel concluded that Wolf Creek Dam exhibited serious problems and that compelling foundation seepage issues required immediate attention. Reports of seepage has caused public worry and showed that if Wolf Creek Dam failed, the USACE will suffer an estimated $3 billion in property damage in such an event. An economic analysis of the new rehabilitation works had suggested 7:1 benefits/cost ratio. All this, triggered the most complex dam foundation remediation project of any dam in the world excluding the case of Mosul Dam foundation works (Iraq). The total cost of estimated for Wolf Creek Dam was $594 million, requiring six years of construction (2007-2013).

In Mosul Dam, the rehabilitation of the grout curtain alone had continued from 1986 up to now costing billions of dollars including 2.0 billion dollar program in 2016-2018 alone.

The new remediation works of Wolf Creek dam was again divided into two phases. The works of Phase I were awarded to Advanced Construction Techniques (A.C.T.) and work on this phase began in March 2006. To reduce stress on the Wolf Creek Dam and therefore risks during the remediation works the Cumberland Lake water level was lowered by 25 meters below the normal summer levels and the project was considered the number one top priority in the United States at that time.

The scope of work consisted of a double line grout curtain to depths of 330ft within the limestone foundation material. The purpose of this grouting was to reduce the residual permeability of the dam foundation as a pre-treatment for the proposed new diaphragm wall installation. The activities also required construction of a 4,000ft long work platform and environmental collection and treatment ponds to support the drilling and grouting operations. The sequence of work required drilling through the embankment overburden and installation of permanent casing up to depths of 180ft seated into the embankment rock interface utilizing sonic drilling methods. Continuous sampling and logging of the embankment material were performed to determine clay core integrity. Site characterization was determined by exploratory drilling using rotary diamond core drilling with continuous sampling and logging of rock cores together with water pressure testing and video imaging. Foundation rock drilling was performed with water actuated down the hole hammer followed by borehole washing, permeability testing and pressure grouting using balanced stable high mobility grouts. Special procedures and grout mixes were developed to treat karst solution features encountered within the rock foundation. Gallery drilling and grouting using rotary diamond drilling, water flush, permeability testing and pressure grouting was also required. IntelliGrout®, a proprietary computer monitoring system was utilized for real-time electronic monitoring and recording of the permeability testing and grouting [9].

In April 2009, construction began on Phase II which was to build a second diaphragm. Plans were developed for this new cut-off wall to be deeper, wider, and to be immediately upstream of the first one. Main contractor for the works was Treviicos-Soletanche JV. Construction commenced in 2006-7 with a scheduled completion target of late 2013.

The reservoir was temporarily lowered to reduce risk until completion of the diaphragm wall. The water level was kept in this case 13.1m below the crest elevation of 220.4m a.s.l. sufficient to satisfy the minimum “power pool” for hydroelectric power generation. The new diaphragm was designed with a minimum thickness of 610mm as measured across the three types of construction employed which were overlapping panels, secant piles, and a panel-pile combination. It was excavated down through the clay embankment to sit in the top 610mm of bedrock. The panels were 1.83m wide by 2.8m long, and overlapped at least 127mm. Concrete was placed by tremie, working up from the base of the excavation and displacing the slurry support. Compared to the era of the first cut-off wall, the new deep foundation project was able to call upon more advanced construction systems, and data monitoring and analyses.

In total, the new wall was designed to be approximately 1158m long and up to 83.8m deep penetrating up to 29m into the bedrock, making it up to 22.9m deeper than the original cut-off wall which has a variable bottom profile. The diaphragm was completed in 2013 and lake levels returned to normal in 2014 [10].

The diaphragm work of Wolf Creek dam led Donald Bruce, the well-known grouting and diaphragm construction specialist to propose one theory which states that advances in specialty geotechnical construction techniques are not gradual and progressive. Rather they take the form of "Great Leaps" triggered by specific project challenges. He qualifies as a "Great Leap," the deep remedial cutoff completed at Wolf Creek Dam, which has given good test to this theory [11].

All the proceedings, however, prove one important fact that geotechnical problems themselves do not arise as "Leaps", but rather can either take many long years of operation to show their manifestations, or immediately on the first impounding of the dam as had happened in Mosul Dam mentioned already. This dam completed in 1986 was built on poor foundation containing gypsum-limestone karst formations. Construction of the grout curtain was met with considerable difficulties upon the refusal of the gypsum breccia layers to accept any form of grouting. The first impounding of the dam with open widows in the curtain led to increasing seepage flow under the rising hydraulic head of the reservoir and causing increasing dissolution of gypsum which had threatened the stability of the dam. Remedial grouting works have been ongoing since 1986 up to now except for the period between June 2014 to April 2016 when the grouting works and all other activities were suspended at the site for fear of ISIS attacks on the site. But luckily resumption of these works was possible later on in a new intensive phase with $2 Billion cost and completed in 2018. The grouting is deemed to continue for the whole life of the dam. This is probably the longest and more costly foundation remedial grouting work ever undertaken all over the world up to now. The dam still forms a threat to six million inhabitants in the downstream waiting for a permanent solution, which may be in the form of diaphragm or constructing a protection dam downstream. The full story of this dam was summarized by three papers [12], [13] and [14].

4. Aging problems in embankments of fill dams

Degradation in the structural properties of fill dams of any type may be due to actions affecting their embankment as result from external forces, or from the nature and properties of the material of which the embankment is constructed and from procedures used in its construction. Such deterioration can also be caused even by improper design. With the passage of time signs of decay may in many cases appear after long years of operation while they had escaped the attention of inspectors during past years. Constant vigil over dam safety and concentrating on even the smallest related details and taking them seriously may save the dam by taking timely actions. Old dams can be more vulnerable to the development of defects as result of accumulation of causes and magnification of impacts during the long service life. The outcome may show as deformation, loss of strength, pore pressure increase, internal erosion, etc. which all contribute to less stable dams.

4.1 Embankment deformation

Earth fill embankments undergo long process of consolidation which starts with the compaction of the fill materials during construction resulting in “Initial Consolidation”. Consolidation, however, continuous for a very long time during the operational life of the dam and it is known then as “Primary Consolidation”. Primary consolidation will slow down after these years, subject to initial degree of saturation of the fill material, type of materials and their mineral composition in addition to mode of operation and ambient environmental conditions. Final stage of consolidation is called “Secondary Consolidation” which is taken to mean all of the compression that follows the “Primary Consolidation”.

Consolidation is caused by the dissipation of pore water pressure in the clays and the continuous settlement of all materials forming the body of the embankment [15]. Although most of the settlement occurs in the first few years after end of construction as result of primary consolidation, more but little settlement will continue during secondary consolidation. The result of these accumulated settlements may show in the form of progressive cracking and deformations which can be uneven along the dam length and may lead also to differential settlement in adjacent sections or in contact sections with abutments and concrete structures causing more cracking and fissuring. This cracking will lead to increased seepage and possibilities of internal erosion with the passage of time. Cracks and fissures can develop severely and become real threats if the dam is subject to sudden settlements from seismic ground movement in earthquake events. Generally speaking, cracking may be caused by differential foundation settlement, differential embankment compression, embankment arching “Hydraulic Fracture”, separation in the contact with concrete structures, or drying out of the upper portion of the core during prolonged dry periods. Loss of freeboard is another side effect of large settlements, which endanger dams’ safety if this was not anticipated and taken care of in the design stage.

4.2 Improper construction

Inferior construction procedures and not following strictly proper specification can lead to potential threats which appear later on after the passage of appreciable length of time. Wetting of improperly compacted embankment soil can lead to loss of strength. Materials compacted dry without the necessary compaction effort to sufficiently reduce the void volume will experience considerable particle reorientation and consolidation, and if a large mass of embankment are affected this results in large differential settlements. If occasional layers or lenses of different material are present in the fill, then such layers or lenses will show different behavior than the general mass which reflect in different compression and may lead to cracking and differential settlement within the dam itself and possibly lead to piping and areas of different pore pressures. This demonstrate the importance of using uniformly homogenous materials throughout the clay core since variation of these material properties can result in erratic behavior of the soil mass. Using different materials of different characteristics in different horizons may cause separation of layers which leads to hydraulic fracturing. Such case was partly blamed for the Teton Dam (Idaho-USA 1973) failure.

4.3 Loss of strength during operation

Many scenarios may be visualized regarding such loss of strength with the passage of time and mean lesser stability in the end. These may be one or more of the following:

i. Some dam materials can experience large reduction of cohesion as in clay cores, and reduction of the angle of internal friction in granular as in shells when they are continually wetted. Reservoir seepage, intensive rainfall wetting the downstream shell and very frequent draw down of the reservoir affect stability by reducing the aforementioned shearing strength parameters and the factors of safety against slope slides originally calculated. From the fact of their long lives, old dams may face critical situations due to unanticipated loading conditions such as greater than the anticipated floods, or in case of unaccounted severity of earthquakes. These unanticipated conditions combined with reduced factors of safety causes great stability concerns over such dams. The mechanical characteristics of fill materials and thereof the calculated design factors of safety cannot be guaranteed over such long time for old dams.

ii. Loss of strength may be caused by change in the state of stress within the dam with increasing pore pressure. Any increase in pore pressure in clay core results in pressures greater than estimated from normal permeability tests of the compacted soil and this is generally associated with opening up of transverse cracks in the core or through the whole fill. Pore pressure increase may be associated also with such matters as dissolution of dispersive clays, defective layer in the core, poor material placement, bad design of the filter and drainage zones which result in low permeability downstream of the core. Good design and construction practices may preclude excessive pore pressure increase by the appropriate selection of graded filter material and careful placement of filter zones and drainage layers. Moreover, eliminating foundation overhangs, and shaping the contacts with the abutments and foundations, use of ductile material in the upper part of the dam and adequate drainage downstream. Watching for any abnormal pore pressure increase is done by using pore pressure measuring cells and observing the development of this phenomenon. Generally, in normal cases these cells show decrease of pore pressure with time as excess pressure is dissipated gradually through the primary consolidation process. Any sudden increase should point out to unhealthy condition which requires immediate attention to pin point the reasons and find a proper way of treatment in time. In many cases it is customary to observe sudden surge in pore pressure after an earthquake. The magnitude of this surge depends on the magnitude of settlement caused by the earthquake which result in more consolidation and also the opening up of cracks. Careful study and analysis after such event is necessary to evaluate the general condition of the dam. Older dams are more prone to such influences with progress of time.

iii. Embankment heightening of an earthfill dam may stress the existing material beyond its peak strength and a lower residual strength is reached. Such heightening is required after the need of such increases due to increasing demand on water brought about after long period since the dam was commissioned. It is advisable that such work is not attempted before careful study of the old dam material characteristics to ensure not stressing them beyond their tolerated capacity.

iv. Cycles of drying and wetting of high plasticity clays may cause in slope instability and sliding failure particularly shallow downstream slips. As the clay dries, capillary stresses lead to cracking by surface tension. When water is again available to the crack, material sloughs off into the crack and there is a loss of strength in the swelling clay at the crack face.

A progression of these events during the long life of an old dam would reduce the effective dam width and promote other scenarios.

4.4 Internal erosion in embankment dams

Internal erosion is one of the most dangerous threats to fill dams' stability. This may go unnoticed for a long time, but generally finds its origin in design and construction inadequacies. It may occur in dispersive soils susceptible to piping and cracked soil mass in the embankment. Internal erosion may happen within the embankment core or in the downstream shoulders if the phreatic surface daylights above the toe of the embankment. Quite frequently it occurs at the embankment contact with the foundation as the case was in Teton Dam failure. Erosion of embankment material may be through fissured rock or into solution cavities along with erodible rock fissure infilling within the dam base and abutments which enhance seepage flow. Erosion of embankment is commonly via embankment cracking and inferior protection by filters. Characteristics and origin of the material used can help in internal instability of the earth fill. Use of glacial moraine till, dispersive clays, or leaching of soluble minerals exasperate the erosion process if it starts. Removed of material by erosion induces settlements and local failures occur as sinkholes. Detection of internal erosion has relied on visual inspection, water flow measurements, pore pressure readings, and turbidity measurements. Drilling using sonic drilling or other drilling method without drilling fluid, geophysical investigations such as temperature measurements and magnetic resonance imaging or by the use of geo-radar have been carried out to determine the extent of the erosion. Remediation of such problems can save the dam if they are discovered at an early stage of its development. Measures to be taken depending on each case may include grouting the piping path, diaphragm wall installation, drain system replacement and replacement of affected embankment section. Progression of internal erosion is characteristic of dams after appreciable service time.

4.5 Mechanical degradation of embankment materials

This implies changes of the physical properties of the fill materials which result in change of strength or increased permeability of the material in question. These can result from:

i. Increasing seepage which is an important element for the degradation processes as it may cause removal of erodible, soluble, or dispersed material. Saturation may cause loss of shear strength.
ii. Some rock types, particularly shales, deteriorate when exposed to air and moisture in an unconfined condition. If shales are used in a rockfill, they may deteriorate or weather into a soil resulting in surface deformation or lowered shear strength of the rockfill allowing slope instability.

4.6 Surface erosion of embankment slopes

Surface erosion, although a very common aging scenario, has not been an important contributor to embankment failures. Surface erosion is readily detected by routine visual inspection and repairs would be undertaken in a timely manner. Erosion on the downstream slope and crest may be due to heavy direct rainfall or surface water runoff, brief crest overtopping, wave spray over a wave wall or wind driven wave spray. Erosion on the upstream slope may be due to wave action on too small riprap or from inadequate bedding, breakdown of riprap or even from freezing and thawing displacement. Timely repair of upstream and downstream slope erosion is important. Downstream slope improvement includes directing crest runoff into the reservoir and collecting and diverting surface water down and away from the downstream slope. Regular visual inspections particularly following unusual weather events are necessary.

4.7 Burrowing animal impacts on earthen dams

The damages caused by beavers, rodents, and other forms of borrowing animals over a long period of time often appear as minor and small burrows, shallow dens, however, may lead to progressive erosion. Limited erosion may not trigger a dam owner’s concern, but in many reported cases minor damages such as these are often at the core of unsafe dam operations or outright failure specially of old dams. The most significant and often least obvious impacts of wildlife intrusions may be described in the following:

4.7.1 Hydraulic alterations

Burrows on the upstream and downstream slopes can dramatically alter how a dam controls the water pooled behind it. Dramatic changes to the designed hydraulic function of a dam include:

i. Shortened seepage paths.
ii. Increased seepage volumes.
iii. Increased probability of slope failure.
iv. Internal erosion of embankment materials (piping), which can rapidly lead to failure of the dam.

In addition, beaver mounds may block principal and emergency spillways and riser outlets, resulting in,

i. Increased normal pool levels and reduced spillway discharge capacity.
ii. Sudden high discharges from the dam if an upstream beaver dam fails.
iii. Clogged water control structures as debris from an upstream beaver dam floats downstream and,
iv. Erosion of the downstream toe as a result of elevated tailwater caused by beaver activity in the downstream reach.

4.7.2 Structural integrity loss

Many species excavate dens and burrows within embankment dams, causing large voids that weaken the structural integrity of the dam. Typical voids can range from the size of a bowling ball to a beach ball and much larger, and can cause many adverse structural impacts. Localized burrow collapse can occur due to heavy rain and snow melt or by heavy equipment or vehicle use on the crest. Collapsed burrows can progressively lead to sinkholes or depressions on the embankment surface. Collapsed crest soils can result in loss of freeboard, thus endangering the dam during storm events. Massive slope instability can result from collapsed burrows and soil losses.

4.7.3 Surface erosion

Wildlife that graze or traverse areas of open vegetation associated with dam embankments can cause a widespread loss of vegetative cover.

This increased feeding and traffic pressure on the dam’s vegetative ground cover can lead to:

i. Erosion paths.
ii. Decreased soil retention on the dam’s crest and slope.
iii. Increased rates of soil erosion because of the lack of stabilizing vegetation from grazing and trafficking.
iv. Irregular surface erosion and the formation of rills and gullies, and,
v. Reduction in freeboard and loss of cross section, and in turn, an increase in the dam’s vulnerability to damage from high water during large storm events [16].

4.8 Loss of bond between concrete structures and embankment, and damaged complementary materials

The deterioration of embankments adjacent to concrete structures is associated with differential movements in the contact zone. Displacements of the bonding zone may be due to settlement of the embankment material caused by inadequate compaction or internal erosion. Settlement of the foundation due to inadequate treatment may also be another cause. Settlements may lead to arching in the fill and a reduction of the effective stress. Moreover, seepage develops through cracks in the fill or along the concrete structure promoting internal erosion and failure if the filter-drainage system is insufficient. Aging of complementary materials used in dam construction, such as geosynthetic materials, asphalt concrete facing, soil-cement protection which are subject to weather elements and stresses due to settlements, abrasion, and ware due to moving debris and other agents can cause serious damage to these materials and may speed up the aging process of the dam itself and contribute to safety problems in such dams [17].

5. Lewisville Lake Dam an aging earth fill dam case study

One example of progressive deterioration in earth fill embankment dams with passage of time may be cited from the case of Lewisville Lake Dam in Texas (USA) completed in 1955 by the US Army Corps of Engineers. This case also shows the continuous concern over its safety expressed by the Corps who have been responsible for the dam's safety since then and considered it in 2005 to be very high risk dam among their more than 700 dams according to risk assessment study. This risk assessment merits consideration of the long term.

The objectives for creating the Lewisville Dam lake reservoir were flood control, serving as water supply source for Dallas City and its suburbs, but residents also use it for recreational purposes. The main components of the project are uncontrolled concrete ogee weir spillway structure and earthen embankment section. The ungated spillway is located on the left abutment of the embankment. The structure consists of an approach channel, uncontrolled concrete ogee weir with a crest elevation 532.0 feet above mean sea level, two 90 feet long concrete gravity non-overflow sections, paved apron, retaining walls, and an unlined discharge channel. The reinforced concrete apron slab is about 209 feet long and 551 feet wide. The slab was built with a 10 feet deep turned-down cutoff at the end of the original apron slab. The reinforced concrete cantilever retaining walls are about 13 feet high and extend the length of the concrete apron. Twenty-eight foot high wing walls are located at the downstream end of each retaining wall. The spillway is designed to pass a discharge up to 6136m^3/sec. The spillway has no stilling basin, and, because of high velocity discharges it was severely eroded at the downstream end of the sill during the uncontrolled releases of 1981 and 1982.

Repairs were made to the spillway and downstream riprap in many occasions between 1966 and 1988 which had included extending the concrete apron 60 feet downstream with 18 inch reinforced and anchored concrete slab and 10 feet deep cutoff wall at the downstream end of the new slab. Additionally, 24 inch thick riprap was placed at the downstream end of the new slab for a distance of approximately 30 feet. Much of this riprap was redistributed during the uncontrolled releases that followed heavy rains in May 2015. Repairs to restore the original riprap design were done as part of normal operation and maintenance activities.

View of the Ogee weir, looking east is shown in Figure 12.

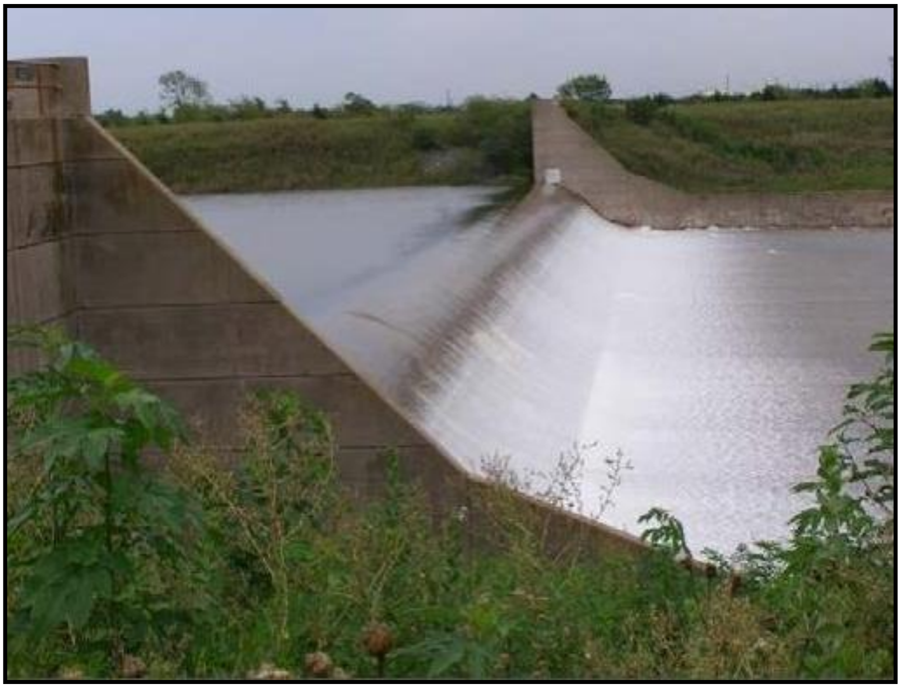


Figure 12: Lewisville Dam: view of spillway Ogee Weir looking east.

The earthen dam is 32,328 feet long, with a maximum height of 125 feet and crest of 20 feet width. The dam was designed as an impervious structure and it was built using materials obtained from onsite borrow sites and excavations. Also incorporated into the design of the dam was 3 to 4 foot thick granular drainage blanket underneath the downstream section of the embankment. The foundation is made up of clayey sands and sandy clays [18]. The dam holds 6.85 x$10^{9}$$m^{3}$ of water when the lake is full to its maximum depth of 20 meters. Aerial views of the dam are shown in Figures 13 and 14 indicating the extent of the dam length and some of the details of the inhabited areas downstream.

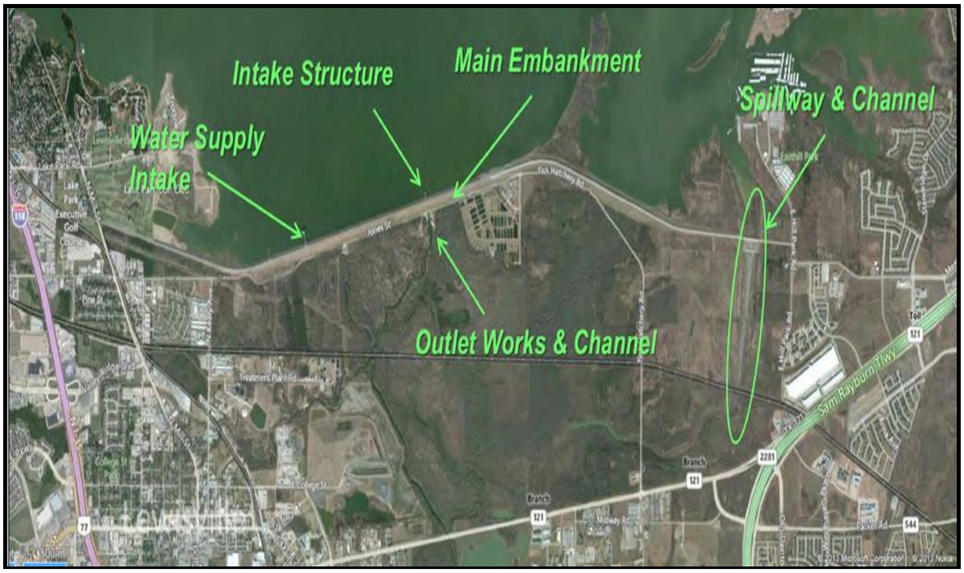


Figure 13: An aerial view of Lewisville Dam indicating the various components of the dam.

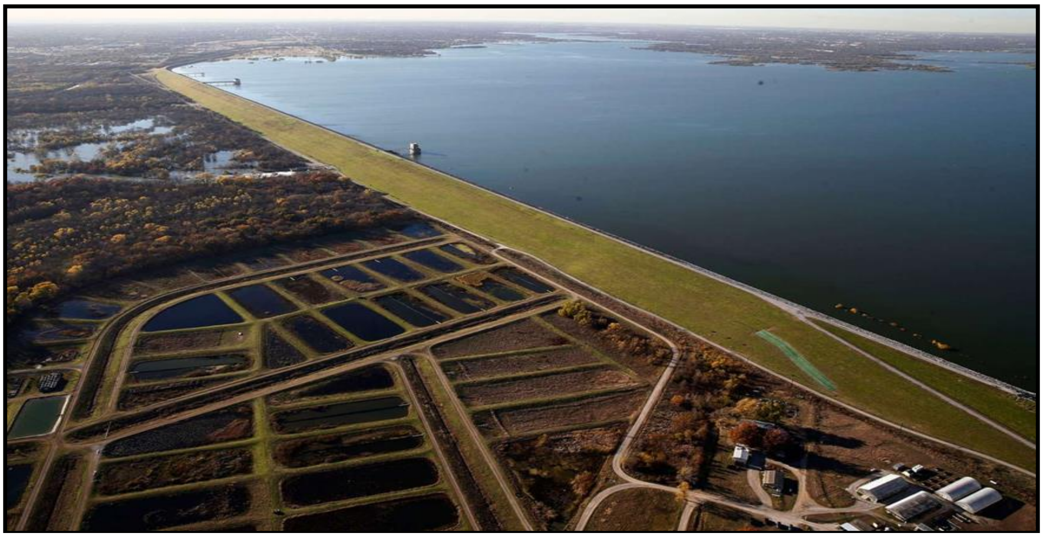


Figure 14: An aerial view of Lewisville Dam showing location of boggy area created by seepage.

A typical section of the dam is shown in Figure 15 which illustrates type of materials and details of the dam while Figure 16 is a schematic diagram which indicates the operation water levels and shows also the type of geology under the dam which can be very much inductive to seepage problems in the downstream. With this type of geology and in the absence of seepage control measures in the foundation, other than the shallow cutoff trench, it is not surprising therefore that the dam had suffered from seepage for many years, whereby three seepage areas were identified in the downstream of the dam.

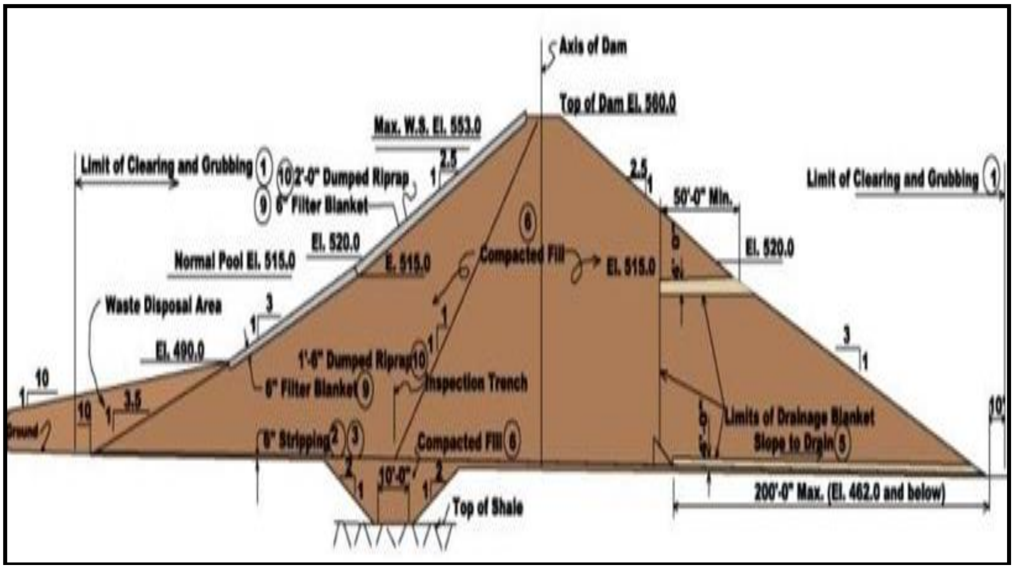


Figure 15: Typical cross section of the Lewisville Dam Embankment.

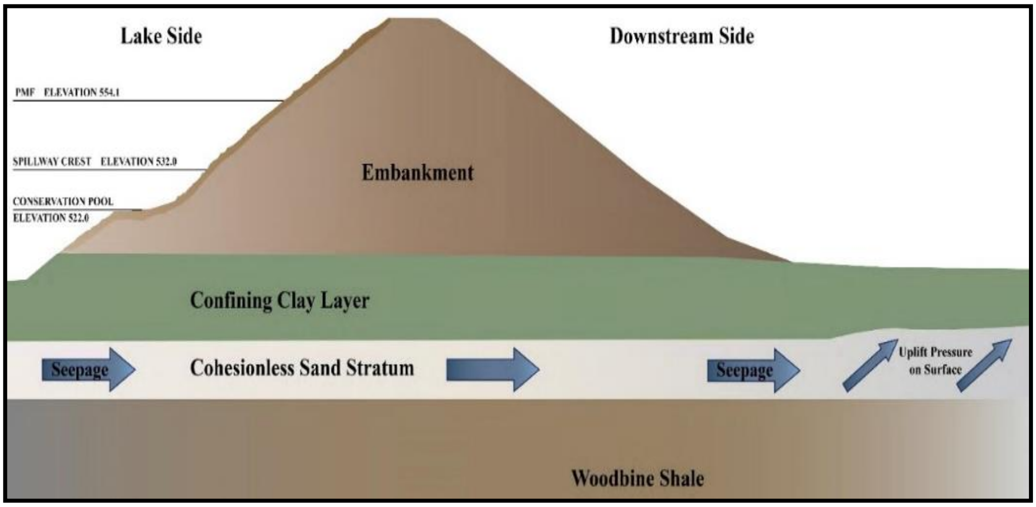

Lake Side
Downstream Side
PMF ELEVATION 554.1
SPILLWAY CREST ELEVATION 532.0
CONSERVATION POOL ELEVATION 522.0
Embankment
Confining Clay Layer
Seepage
Cohesionless Sand Stratum
Seepage
Uplift Pressure on Surface
Woodbine Shale

Figure 16: An exaggerated schematic diagram showing Lewisville Dam Embankment cross section and foundation geology.

The outlet works of the dam consist of an intake structure, conduit portal unit, stilling basin, a service bridge, and reinforced concrete wing wall located on the upstream end. It is equipped with three flood-control openings controlled by three service gates. Two reinforced concrete service conduits go out of the intake structure, each of them is 4.87m in diameter and 135.6m long and equipped with valve at its end. Each of the valves is located in a conduit portal unit at the downstream and used for diverting low flows either to the stilling basin, or to the penstocks for the non-federal hydropower facility. The stilling basin consists of a parabolically curved, reinforced concrete apron with the slab and training walls. The stilling basin is 54.8m long and is approximately15.8m wide at the end sill and provided with wing walls at the downstream end.

The dam began to show signs of severe deterioration at an early date after commissioning. But in 2008, a group of Corps engineers performed an in-depth assessment of the dam safety and discovered some serious defects; seepage under the foundation was creating pressure and uplift conditions at one end of the dam. There were signs of embankment instability during “extreme loading conditions” under rapidly rising reservoir. The spillway suffered from erosion and structural distress. The same team concluded the “likelihood of failure from one of these occurrences, that this likelihood was too high to assure public safety”, and, the dam posed “very high risk” to the population centers downstream namely: Lewisville, Coppell, Carrollton, Farmers Branch, Irving, Las Colinas, Dallas and points south. The Corps had prepared maps of the inundation area which showed that 431,000 people will be at risk. One general map of the will be inundated area is given in Figure 17.

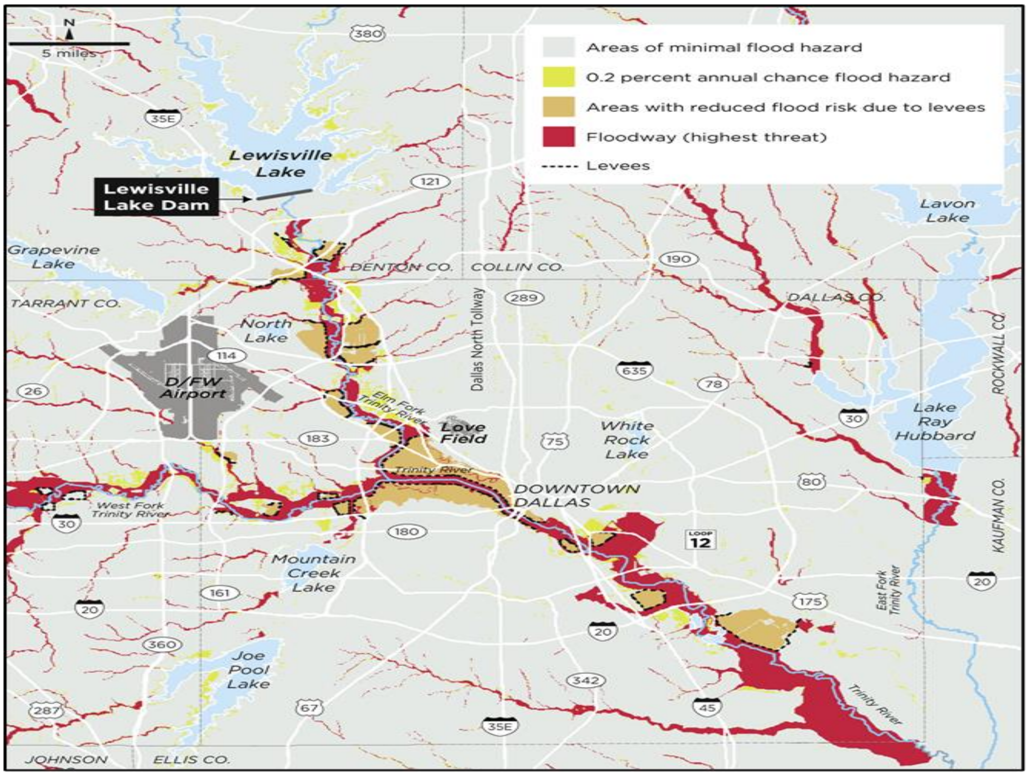


Figure17: General map of the inundated area downstream of Lewisville Dam in case of dam failure.

The analysts' findings were supported by another study done by another team of engineers who identified significant potential failure modes with respect to the project condition and the potential downstream consequences.

Worries about the safety of the dam continued during the following years and in a public meeting held in Lewisville in 2013, Corps officials acknowledged the uncontrolled seepage, the potential instability of the spillway and embankment and other areas of concern. But they assured the public that the Corps had implemented a number of temporary risk-reduction measures to address the problems while pressing ahead with further studies to determine the best methods to fix them [19].

As events unfolded later on, it was in 2015 during the height of the record rain storm in May 2015, as floodwaters began filling the Lewisville reservoir, the safety inspection team kept a vigil to observe signs of distress for about six weeks in May and June. The higher the water rose in the reservoir, the greater the pressures on the Lewisville Dam became. So, when an instrument reading showed pressure rising at "Seepage Area No. 1", the same area where the Corps specialists had worried for years that excessive seepage could cause a rupture or blowout, a sand boil was found in the boggy area which looked like a whirlpool spinning and spouting from underground. This indicated that excessive seepage was starting to erode soil material from the downstream slope or foundation of the dam through to the upstream side to form a pipe, or cavity, to the reservoir in retrogressive internal erosion processes. It was recognized as the beginning of piping failure and internal erosion; the structural integrity of the dam was already compromised and complete failure of the dam was threatened. An emergency action was initiated by placing one-layer ring of sandbags around the sand boil allowing the pool of water inside to rise about six inches. Such action is normally taken to equalize the pressure and prevent the water and soil particles from eruption.

At the same time emergency coordinators prepared to begin evacuations in their communities. But the sandbagging worked and the boil, which had surfaced May 17, subsided in a matter of days. And the evacuations were called off and major catastrophe was averted.

Troubles at the Lewisville Dam, however, were far from over. On June 23, 2015, an extensive slide on the upstream face of the embankment,160 feet long and 23 feet wide occurred. Fortunately, this was not a deep seated slide and it did not extend into the foundation of the dam. The location of the slide was adjacent to an earlier major slide that happened in 1995 suggesting to the engineers' instability within the embankment.

The photograph in Figure 18 shows the slide which was covered with plastic tarps, sandbags, and wooden planks for fear of more heavy rain causing more damage. Longitudinal cracking at the crest edge as the one shown in Figure 19 had also existed for some time and the cyclic saturation of the upstream face due to rising and falling water surface had induced failures in the saturated mass of soil. Wave action and changing water level due to operational requirement and rapid rise and fall during short floods must have had all contributed to these failures.

It was recognized since the beginning that the 10.0 million cubic meters of fine-grained soil and clay used to build the 10 kilometer long embankment shrinks during droughts and swells during rains, reducing the slope's shear strength, which means the embankment is brittle when it is dry and spongy when it is wet.

The embankment stability was more questionable since the May, 2015 flood because cracks have spread along the 6 meters wide crest of the dam. The Corps believed that "desiccation cracking" at the crest of the dam caused the earthen slope to collapse and slide. The heavy rains in mid-May and early June of 2015 filled the 1 to 3 inch wide cracks, saturating the soil underneath and vastly increasing its weight while reducing its shearing strength had all caused instability and sliding. This dramatic shrinking and swelling of the embankment materials during the rapid drought-to-deluge cycle may also account for another deformity that were spotted on the upstream face of the dam after the May rains which was a large, bowl-like depression in the slope across from Seepage Area No.1.

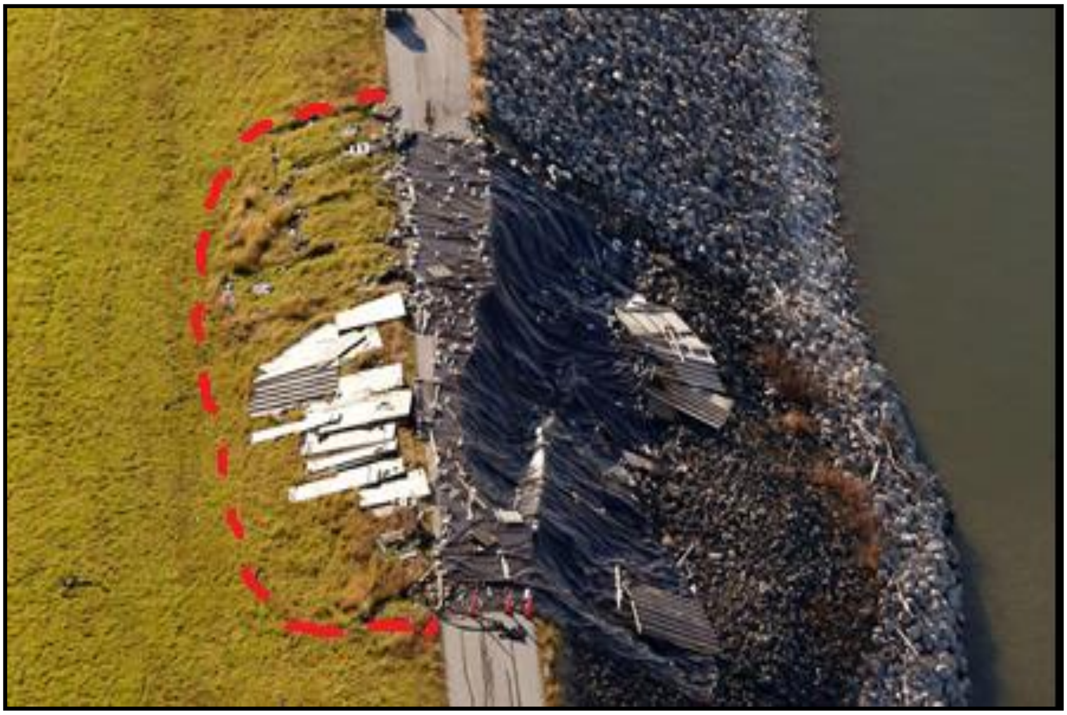


Figure 18: Slide area covered with plastic tarps, sandbags, and wooden planks for fear of heavy rain causing more damage [20].

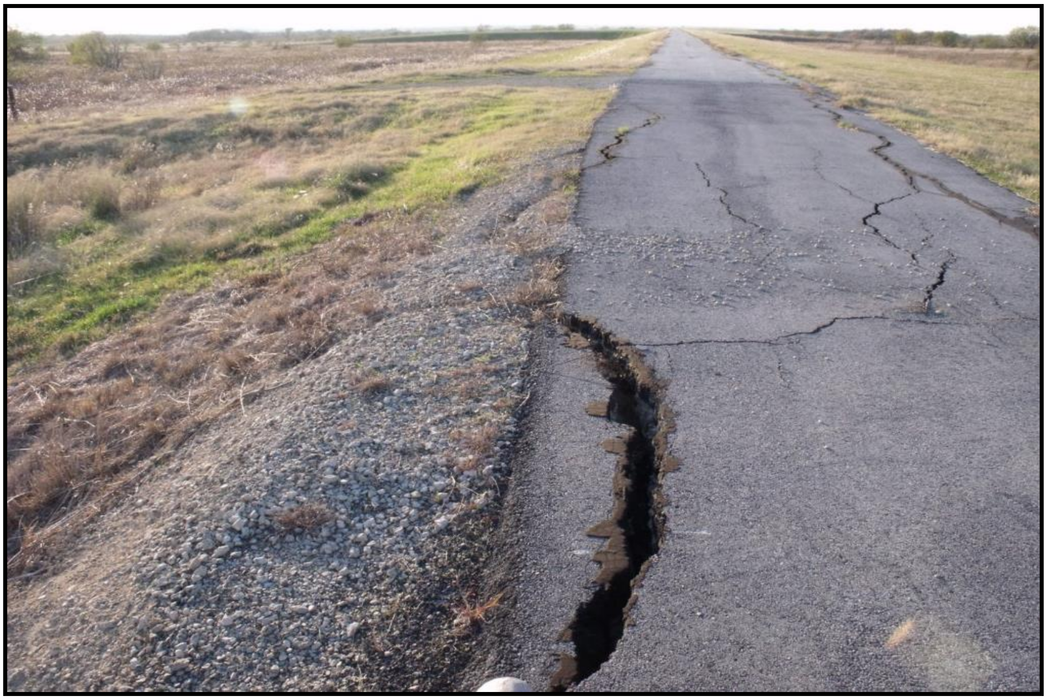


Figure 19: Typical cracking on Lewisville Lake Dam with clear signs of aging.

The envisaged solution was to surround the slide area at the upstream side by a cofferdam, pump dry the enclosed area to enable heavy-duty construction work below the reservoir's waterline. The contract for this repair work was awarded as a $6.4 million contract to repair the slide in late 2015; it was scheduled to begin on 4th Jan, and should be completed by May 2016 [21].

The works went ahead as scheduled, but close to construction completion another small slide of about 850 cubic meters occurred which was replaced also. Since the soil used in the dam contained high amount of clay, which expands and contracts with moisture, the Corps replaced the failed material of the sliding slopes with lime-stabilized soil taken from borrow area downstream of the dam. The progress of work in this case of repair is shown in Figure [20]. But as it was coming to the final stage of adding the slope protection riprap to the newly constructed slope [22].

Occurrence of more of such sliding failures in the dam slopes with the progress of time cannot be ruled out, as the same soil and hydraulic conditions prevails along the whole length of the dam. Yet preventing of another massive slide at the Lewisville Dam may not be as easy as fixing the last one. Taken together, the increased seepage, the boils, the piping, the progression of cracking, the slides, the depression and other signs of serious distress, since the 2015 flood, have all indicated that the work of aging process within the dam was progressing at a high rate which merited serious upgrading works to avoid the high safety risks on one hand, and for the sake of continued benefits of the dam on the other.

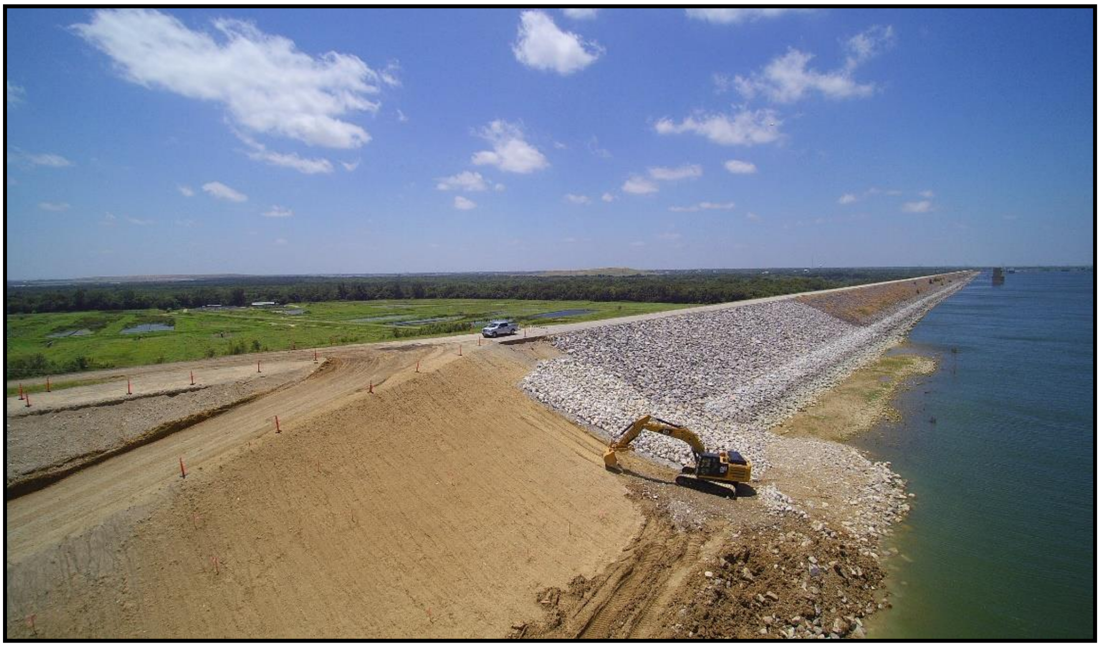


Figure 20: Repair work progressing on the slope of Lewisville Dam slide during 2016 [22].

Major repairs were already done since the project was commissioned which included,

i. Repair of riprap (November1966-May 1967).
ii. Embankment upstream berm modification (July 1979-May 1980).
iii. Embankment and downstream berm (May 1981-November 1983).
iv. Upper slope rehabilitation (June 1983-April 1984).
v. Spillway repair (September 1984-September 1985).
vi. Spillway wall and spillway slab repair (October 1987-May 1988).
vii. Upstream embankment and erosion repair (October 1995-July 1996).

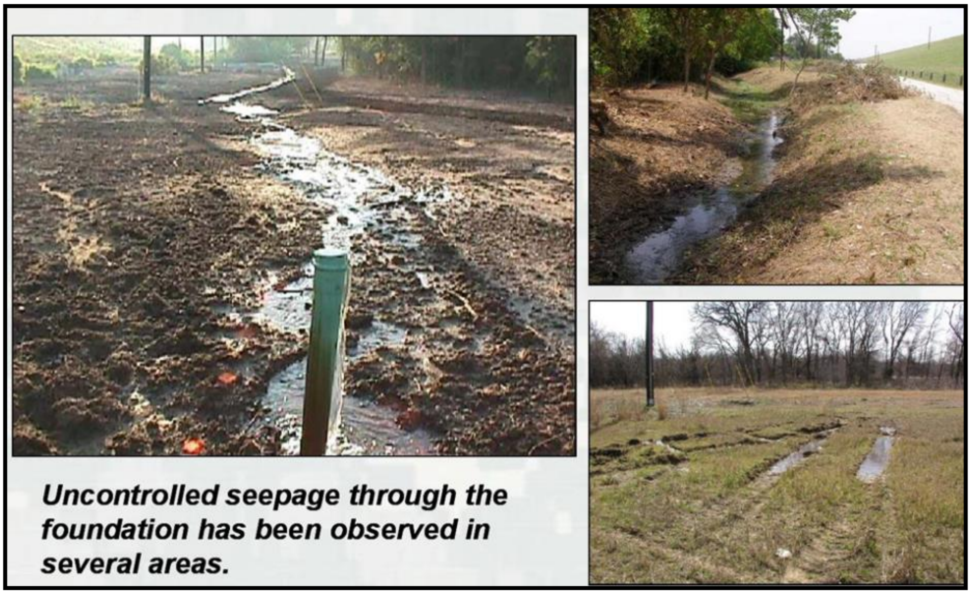
It was recognized also since 2013, even before the damages of the 2015 flood, that the dam's safety was at risk. Studies under the Corps' Risk Assessment Process had confirmed the existence of unsafe conditions and recognized that the actual level of risk within the dam was high [23].

A preliminary study was made at that time to establish areas of concern which was followed by more rigorous risk assessment to show the impacts of such defects and their influence on safety and laying the ground for remedial actions to be taken to reduce them to tolerable levels.

So, in 2015, after the event of May-June flood these remedial actions were elaborated and were presented in a meeting on November 16, 2015 [24].

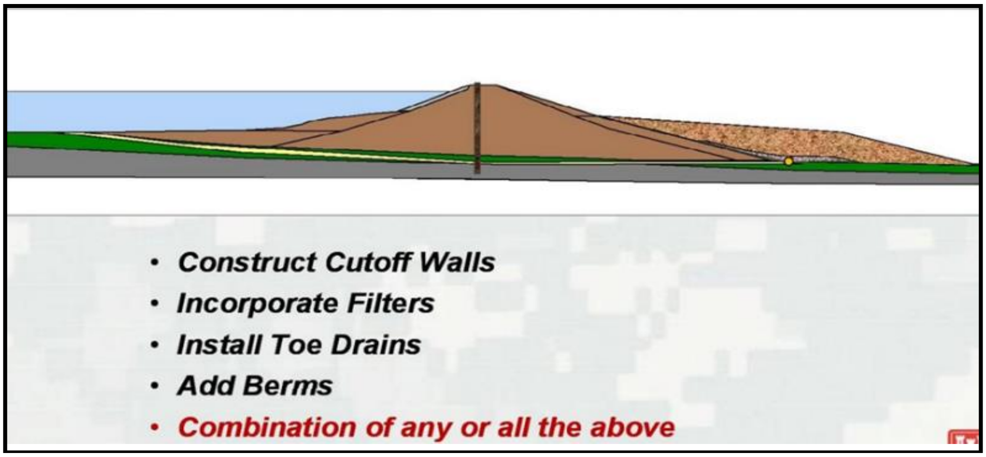
The following safety issues were identified, and the possible actions were indicated in the 2013 and 2015 assessments,

i. Uncontrolled seepage through foundation was creating unsafe conditions which could develop into piping and lead to erosion of the embankment. This seepage had created boggy areas downstream of the dam and even formed some noticeable water streams as shown in Figure 21. Potential corrective measures ranged between adding filters, toe drains, berms and cutoffs, or combination of all of these measures as suggested in Figure 22.



Uncontrolled seepage through the foundation has been observed in several areas.

Figure 21: Uncontrolled seepage water from under the dam found in various area downstream.



- ***Construct Cutoff Walls***
- ***Incorporate Filters***
- ***Install Toe Drains***
- ***Add Berms***
- ***Combination of any or all the above***

Figure 22: Potential corrective measures for seepage problems.

ii. Embankment Stability was an issue of concern under conditions of higher pool levels due to very high floods. The situation can be aggravated in view of the type of soil materials in the foundation and the embankment itself. The loose and brittle nature of the soil is illustrated in Figure 23.

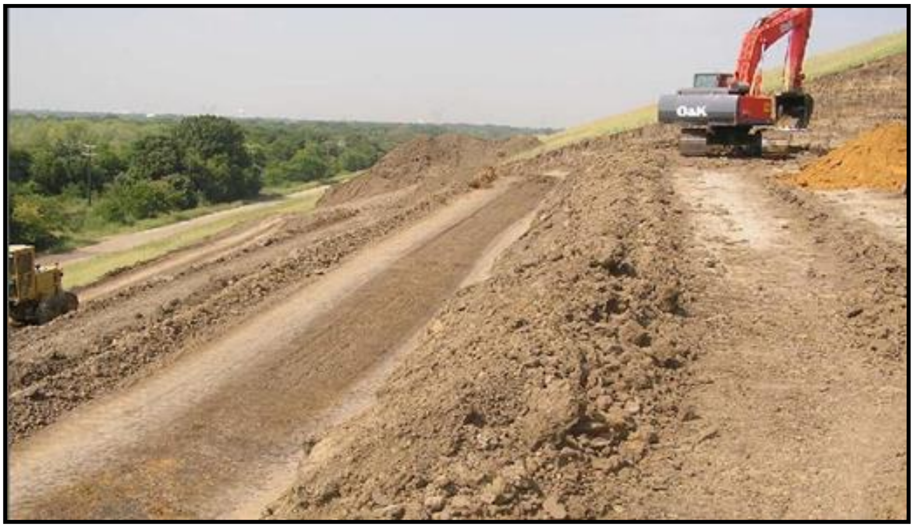
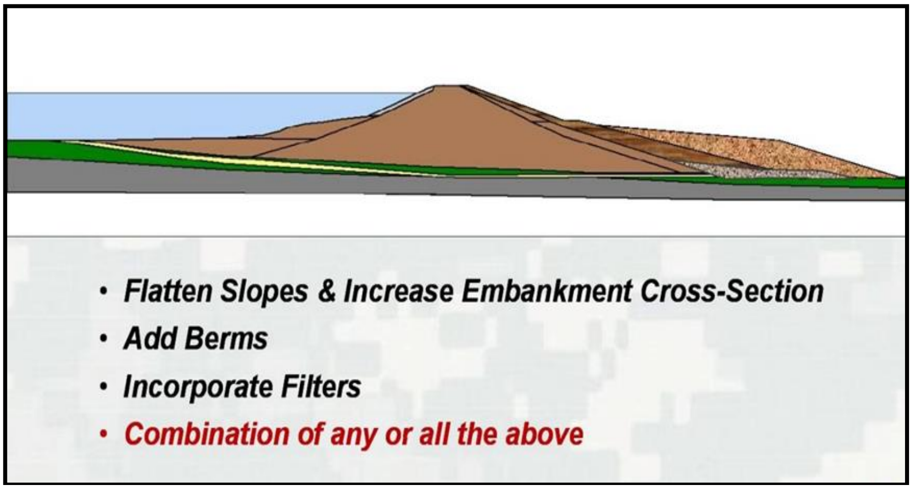


Figure 23: Fine materials used in the construction of the embankment.

Stabilization of the embankment was necessary under the failure scenarios considered. The corrective measures could vary between many alternatives; such as, flattening of the slopes and increasing embankment cross section, adding berms, and incorporating filters, or combination of any or all these solutions, Figure 24.



- *Flatten Slopes & Increase Embankment Cross-Section*
- *Add Berms*
- *Incorporate Filters*
- *Combination of any or all the above*

Figure 24 : Possible corrective measures to improve embankment stability.

iii. Stability of Spillway Weir: Studies showed that the uplift pressures from ground water acting on the spillway weir have the potential to initiate progressive failure of spillway components and the underlaying foundation materials, so potential solutions varied between anchoring the weir and apron by installing four post tensioned anchors through the concrete into the foundation to prevent sliding, or buttress the weir with piers wedged at the downstream end to increase sliding resistance of the structure. Moreover, a geotextile membrane could be laid on the upstream apron to reduce the amount of water that can get beneath the concrete structure and elongate the seepage path reducing uplift.

The downstream spillway apron can also suffer from high uplift pressure and therefore measures to reduce the possibility of sliding and initiation of progressive structural failure must be taken by overlaying the existing slab by a new concrete slab together with installing drainage system between the new and old slabs, or removing the whole apron and install a drainage system underneath a new one Figure 25, shows two views of the spillway during operation (on the left) and another view (on the right) of it while not working and showing cracks in the apron [24].

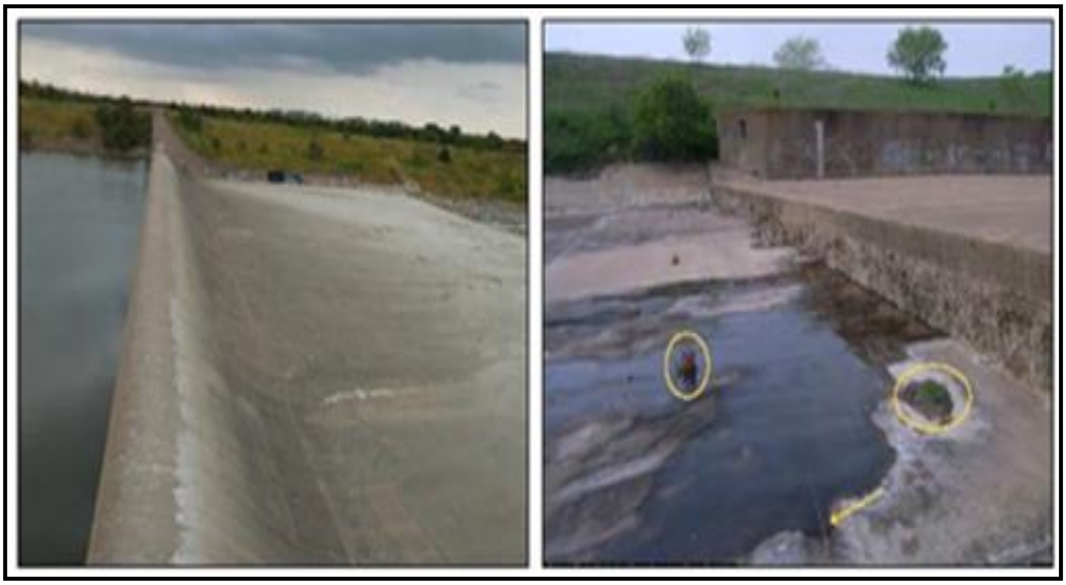


Figure 25: to the left, view of the Spillway when overflowing; to right view of the weir when not working showing cracks in the downstream apron.

iv. Spillway Outlet Channel Instability; Problems of stability were encountered during construction and operation of this earthen outlet channel where many slide collapses had occurred in its the side slopes. These failures were continuous problematic issues during the service life of the dam, and they needed; therefore, permanent solution to enhance their stability. The extent of the damaged reach may be visualized by referring to Figure 26.

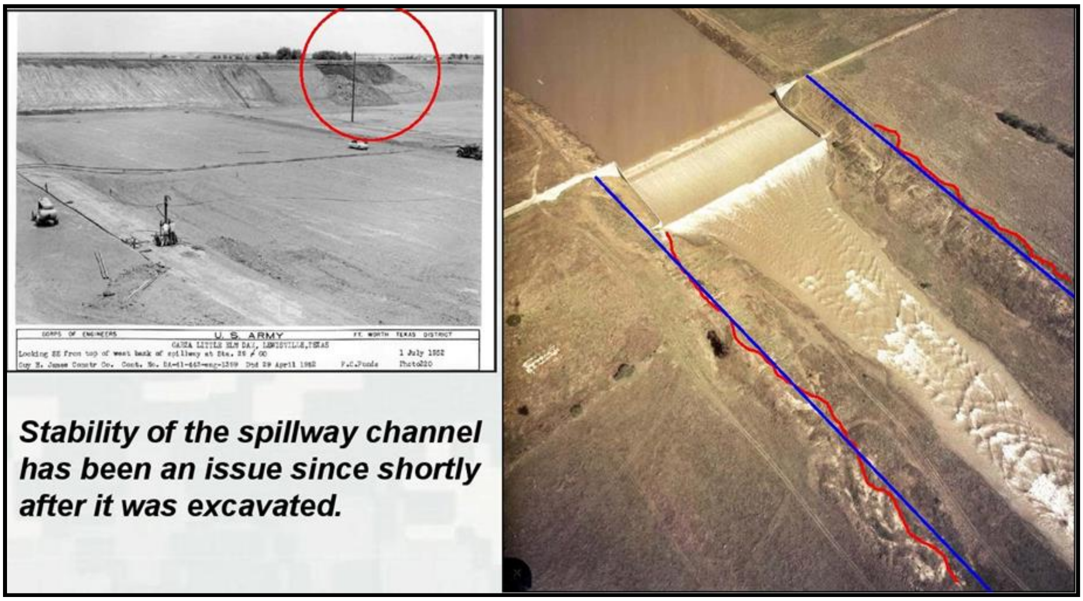

CORPS OF ENGINEERS
U. S. ARMY
FT. WORTH TEXAS DISTRICT
1 July 1952
Stability of the spillway channel has been an issue since shortly after it was excavated.

Figure 26: Stability problems of the spillway outlet channel.

Summarizing all safety issues, their implications on the dam itself and the whole downstream environment with the mitigation measures was done in a study titled "Environmental Assessment Proposed Dam Safety Modification-Lewisville Dam Elm Fork of the Trinity River Lewisville-February 2017".

The study was completed by the USACE to decide on possible future courses of action.

It materialized in the following alternative which were,

i. Take no action alternative; this was rejected for not ensuring the requires safety standards.
ii. Achieving only the tolerable risk limit for life-safety, this was also rejected for the same reason.
iii. Meeting full tolerable risk guidelines; this was the selected alternative, as it answers all safety and environmental requirement and at the same time it is the lower cost one.

The following last two alternatives were not satisfactory for the indicated reasons:

iv. To replace the dam with a new earth fill dam constructed to modern standards to protect against all deficiencies identified, and to achieve the best safety rating applied to USACE dams nation-wide. The USACE did not consider this alternative viable because it was believed that lower cost alternative.

Would effectively reduce risk, therefore, the extra costs associated with this potential alternative are not justified. Furthermore, environmental impacts associated with removing and replacing the entire dam would likely be substantially greater than those resulting from any of the considered action alternatives.

v. To remove the dam; this alternative was not considered by judging it to be not viable because of the resulting annual flood damages and lives at risk downstream in addition to loss of water supply, recreational benefits, the loss of power generation, and the cost of removal. This alternative was not considered as meeting the purpose and need of the Proposed Action.

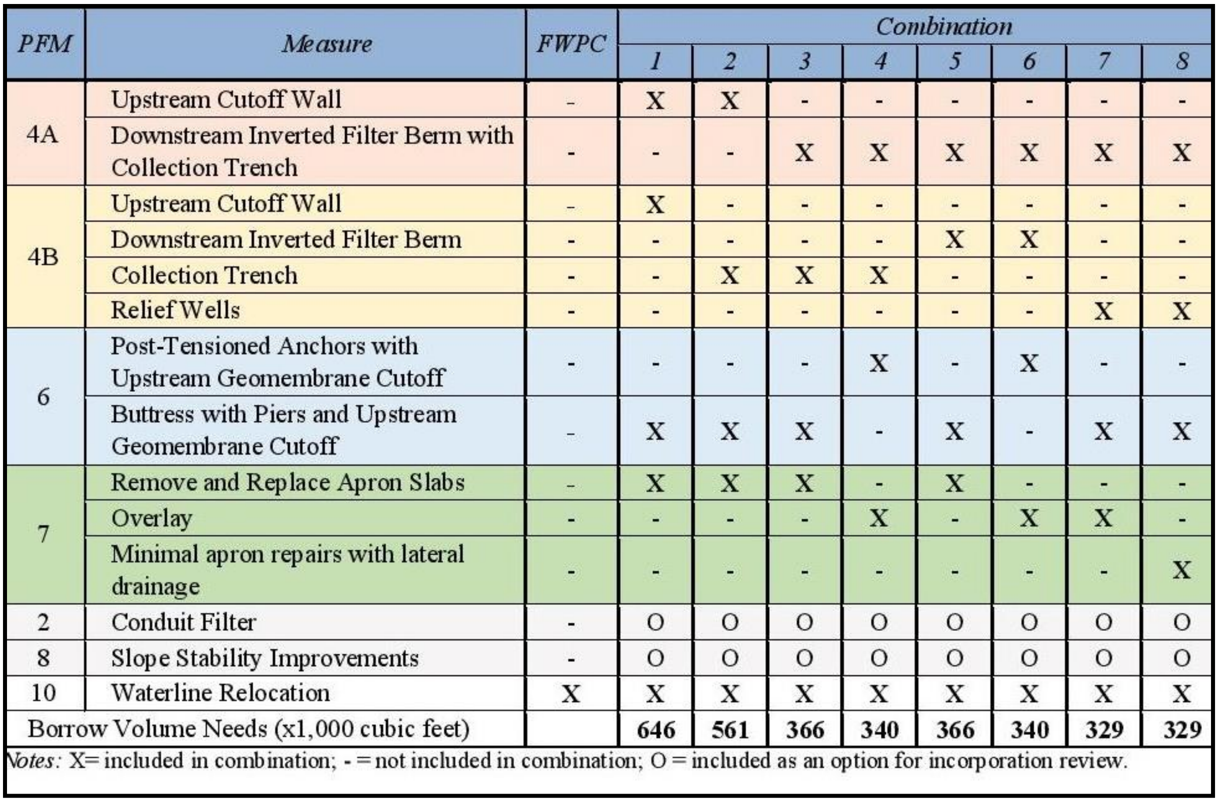
The selected action plan (iii) was actually evaluated after all possible failure modes were analyzed and their drives were investigated.
The failure modes were,

i. Erosion of the embankment caused by seepage. There are three areas with high rates of seepage, two of which were identified as risk-driving potential failure modes (PFMs). Erosion can occur underground if there are cavities, cracks, an unprotected exit, or other openings large enough so that soil particles can be washed into them and transported away by seeping water by piping. The piping and erosion could rapidly progress and erode the dam leading to a complete breach. Water supply pipes downstream of the toe of the main embankment also may provide a potential seepage route. A pervious sand deposit overlying the bedrock and located beneath the embankment has provided a pathway through which clear seepage and undesirable uplift pressures have occurred at the toe of the embankment. In all, the existing seepage control features are not considered sufficient to prevent initiation of a piping failure.

ii. The spillway erodibility potential failure modes. These may result from; First, loss of the spillway crest and partial loss of the reservoir, or; Second, from extremely high water releases over the spillway which causes erosion of the spillway channel downstream of the spillway concrete apron and could potentially shift the apron panels.

The proposed remediation alternative under action plan (iii) was formulated then with multiple combinations of potential remediation actions for each PFM, but using different groups of measures to do so. Eight combinations were developed for further study. These combinations are listed in Table 3.

Table 3: Summary of PFM Remediation Measures Combinations.



PFM	*Measure*	*FWPC*	*Combination*							
			1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
4A	Upstream Cutoff Wall	-	X	X	-	-	-	-	-	-
	Downstream Inverted Filter Berm with Collection Trench	-	-	-	X	X	X	X	X	X
4B	Upstream Cutoff Wall	-	X	-	-	-	-	-	-	-
	Downstream Inverted Filter Berm	-	-	-	-	-	X	X	-	-
	Collection Trench	-	-	X	X	X	-	-	-	-
	Relief Wells	-	-	-	-	-	-	-	X	X
6	Post-Tensioned Anchors with Upstream Geomembrane Cutoff	-	-	-	-	X	-	X	-	-
	Buttress with Piers and Upstream Geomembrane Cutoff	-	X	X	X	-	X	-	X	X
7	Remove and Replace Apron Slabs	-	X	X	X	-	X	-	-	-
	Overlay	-	-	-	-	X	-	X	X	-
	Minimal apron repairs with lateral drainage	-	-	-	-	-	-	-	-	X
2	Conduit Filter	-	O	O	O	O	O	O	O	O
8	Slope Stability Improvements	-	O	O	O	O	O	O	O	O
10	Waterline Relocation	X	X	X	X	X	X	X	X	X
Borrow Volume Needs (x1,000 cubic feet)			**646**	**561**	**366**	**340**	**366**	**340**	**329**	**329**

Notes: X= included in combination; - = not included in combination; O = included as an option for incorporation review.

Adopted course of action was considered over the course of approximately seven years. Implementation of the Action Plan would be divided into two phases:

Phase 1. This phase shall be completed in four years, from fiscal year 2018 to fiscal year 2022, whereby (PFM 4A) and (PFM 10) measures would be implemented first over the course of approximately one year. (PFM 4B) measures would be implemented after the completion of (PFM 4A) measures, and take approximately one more year. But (PFM 6) and (PFM 7) measures would be put into implemented in late 2018, and take approximately three years overlapping with the other previous work.

Phase 2. This phase shall begin in mid-2022 and takes about three more years and it begins with implementing of (PFM 8) measures and takes approximately one year, then (PFM 2) measures begin in mid-2023 and take approximately one year. Grading of the borrow sites shall be done after completion of (PFM 2) measure together with habitat measures would then be implemented and adaptively managed as part of the Lewisville Lake Environmental Learning Area (LLELA) [25].

In a statement issued by the USACE dated on 3rd, May 2017 it endorsed the Modifications Plan as one which would reduce the risks of dam failure to within risk guidelines that are adopted by the Corps and confirmed that the works to start in July 2018 and continuous in phases through mid-2024 [26].

The purpose of presenting this lengthy account on Lewisville Dam is to emphasis that some dams may show signs of deterioration during moderate age, in spite of intermittent repairs, as had happened in this dam, but risk of failure after that progresses quickly and serious safety problems occur, requiring costly repairs. In 2017, the degree of degradation of the dam was confirmed to have reached such a high degree that it posed very high risks to the population and required eight more years of planning and implementation of upgrading with even much more costs.

6. Summary points and conclusions

1. Embankment dams undergo ageing processes as normal for all dams and other manmade structure. This is brought about by the impacts of external forces of nature including the actions of floods and earthquakes events to which the dam is subjected during their long service. Ageing of such dams also progresses consequent to internal actions of the elements forming them and inherent in the nature of the foundations or the materials of the embankments themselves.
2. Recurrent high floods leave their marks on dam structures such as on its appurtenant structures, its side slopes, its surface protections, and increase silt loads and impacts of ice sheets which have cumulative damage even with maintenance. Floods may pause a bigger threat to old dams than more recent dams for the fact that they were designed using limited hydrological records. They may have not encountered the worst possible flood yet to come. This applies also to the methods of structural analysis used when these dams were designed as the more refined methods of today may reveal areas of concern that are exasperated with the passage of time. Maintenance works may face difficult decisions.
3. Foundation problems facing old dams may be attributed to geological conditions not properly appraised and treated in the beginning and show their impacts with the progress of time. Some examples of these situations are existence of faults, presence of karsts, soft rock, soluble rock, and particular ground water conditions. Passage of time will show gradual deterioration of the foundation when karsts are enlarged, clayey material layers or seems are compressed, lenses of soluble materials are dissolved. The outcome of these shows in high total settlement and differential settlements within the dam and in the following cracking in the embankment body. Inefficient foundation treatment or gradually deteriorating seepage control measures such as diaphragms and grout curtains lead to erosion of foundation material, piping, and increased uplift pressure, which all result in undermining dam stability. Outside human activities which can have environmental changes such as oilfield and mining works have also been shown in many instances to create geological hazards with far reaching impacts on dams which appear after long period of time.

It is common to observe deterioration in dams' conditions as they become older and it is familiar to see huge and costly rehabilitation works done to reduce such their hazards. Such situations cannot continue forever either for technical or economic reasons or both and sooner or later the owners will think of decommission them, admitting that budgeting problems may limit the maintenance works especially in dams which have outlived their useful lives.

4. Other than the geological factors, construction procedures, materials used and some outside influences leave their impressions on old embankment and their ageing processes. Such matters are; deformations of the embankment due to the continued consolidation of the fill material, improper construction related to material selection, control of moisture and compaction procedures, pore pressure increase due to use of defective filters can be cited as contributors to problems in older dams. Actions which may lead to unforeseen undesirable results are some things like dam heightening or changing its operation rules without prior appraisal of its current conditions and assessment of future consequences. Moreover, outside impacts of intensive rain storms, borrowing animals, grazing of animals leading to surface erosion and gullying show well in old and neglected dams.

5. Available documented cases of dam ageing and its manifestations are very numerous and rich with details. In this present work only, few representative cases are taken to demonstrate some of the important factors which hit aging dams.

6. The general conclusion drawn is that proper design and good workmanship can elongate the service life of all dams including embankment dams, but the time will come sooner or later when old dams must be decommissioned when their service life expires. In such time economical and technical considerations dictate such decision.

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Dams Safety: the Question of Removing Old Dams

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Abstract

Many old dams in the world today may not be safe enough and represent threats to the communities they serve. They have reached the end of their technical and economic lives making upgrading them questionable. This raises the question of decommissioning or removing them open for further discussion. In this paper the issues related to keeping old dams are discussed showing with one example that the soaring costs of upgrades make it impossible to perform for countries with limited resources without outside financial support. An explanation is also given to show how even in rich countries this is met by budgeting obstacles. Other objections to the presence of these dams, added to the safety question which support of dam's removal are discussed. They include the accumulated damage they have caused to the ecosystems such as siltation and fish migration. An emphasis is put on the need for intensive studies required before removing any such dam in order to mitigate any negative impact subsequent to such removal; and many actual examples are given to illustrate this.

Keywords: Upgrading, Decommissioning, Dam Safety, Ecosystem, Siltation, Fish Migration.

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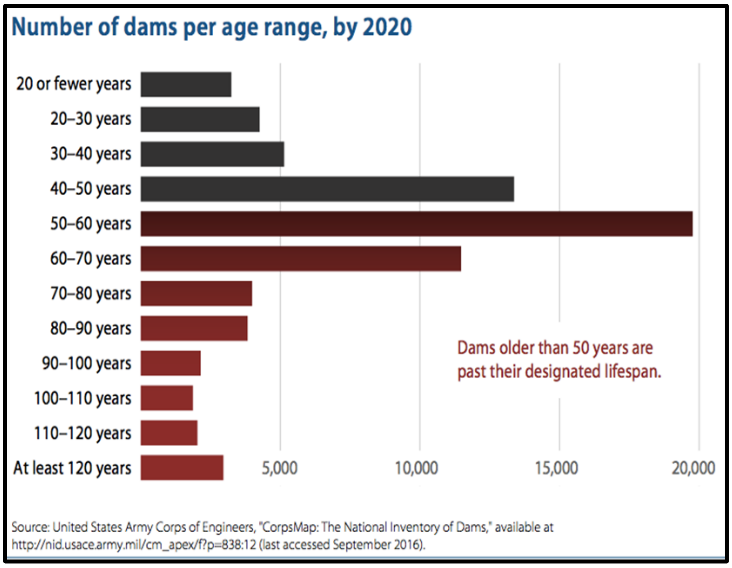
1. Introduction

Dam safety is one of the biggest problems facing societies today. While the need for building more dams is increasing due to increasing needs of growing populations and expanding industries, hazards posed by older dams are aggravated by intensified inhabitation and growth of industry in downstream areas much more than the days when these dams were built. Signs of weakening on these dams add to worries of their possible failure causing loss of life and destruction of infra structures and material property loss. The decision then must be taken, whether to continue spending on repairs, rehabilitation and upgrading of such dams with increasing costs as time goes by or, maybe it is cheaper to decommission them or remove them altogether.

Without any doubt, dams built today are generally more secure than those built 50 years ago, but the majority of dams in the world as a whole is ageing. Around the world, 5,000 large dams are at least 50 years old. Dams in USA are 56 years old on average and there are about 2000 dams of all sizes at least 120 years old, Figure 1. These older dams are highly prone to dam break, especially in countries that lack adequate monitoring and maintenance or funds to do meaningful upgrading and repair. Old dams become increasingly more expensive to maintain with the passage of time. But, this is faced by systematic underfunding of dams' maintenance all over the world. Such maintenance is not considered as a budgeting priority, or it is not carried out properly due to shortage of qualified personnel, and in many instances, even lack of understanding by the higher authorities of the possible dangers.

Upgrading of old dams may be necessary in great number of dams which are fifty years old or more which are still in good shape, but they may be considered unsafe in view of the fact that they were designed using design data and criteria which are no longer applicable.


Number of dams per age range, by 2020

Dams older than 50 years are past their designated lifespan.

Source: United States Army Corps of Engineers, "CorpsMap: The National Inventory of Dams," available at http://nid.usace.army.mil/cm_apex/f?p=838:12 (last accessed September 2016).

Figure 1: Statistics of dams in USA by age (Source: as shown).

Upgrading of such dams may be, neither technically nor economically possible leaving the question to dam owners of keeping such dams and meet the possible hazards, or just take the painful decision of decommissioning or removing them. It is estimated that only in the USA more than $70 billion are needed to rehabilitate the USA dams [1]. Of approximately 84,000 dams in the United States National Inventory of Dams, most are owned by private business, citizens, state governments, and local governments. Many dam owners are unable undertake dam repairs and rehabilitation due to lack of funding. This situation often results in dangerously neglected and deteriorated dams [2].

Accumulation of new data on hydrological events and seismic activity during the past 50 or more years has shown that many flaws do exist in the design of a great number of old dams which call for upgrading measures to be taken making the dams safer. Moreover, great number of large dams, even recent ones, have not been built to allow for the erratic hydrological patterns that climate change is bringing. In this sense, all dams should now be considered relatively unsafe. More extreme storms and increasingly severe floods will have major implications on dam safety [3] and [4]. Dams are not built to serve forever. Today, more communities than ever are considering the option of removing or modifying old dams that have not only damaged local riverine ecosystems, but they pose increasing risks due to their physical conditions and the increased development in their downstream areas, which were not so crowded at the time when they were built. Having all these matters in mind, it is necessary to explore the question of aged dams' safety and their viability from technical and economic points of view and to prob the choice of "Remove or Repair" of dams while at the same time not failing to mention the increasing pressures of the growing Green Movements and Environmentalists to restore natural flow in dammed rivers and minimize the damage to the ecosystems brought about by dam construction.

2. Cost of keeping aging dams

Dam’s owners require to allocate budgets continually to carry out routine maintenance, repairs, and upgrading of safety conditions, in addition to the normal operation costs. While some dams continue to serve their useful purposes, others have reached their planned economic and technical limits. Some dam owners may defer maintenance on the basis of high costs to the point where such dams pose threats to public safety, which may be true for some governments lacking enough funds and so reaching such critical situations. An example is the Inguri Dam in Georgia, which was built in 1975 during the Soviet Union era and represented at the time one of the world’s most ambitious pieces of structural engineering. The importance of this dam is reflected by the facts that it is hydroelectric dam which impounds the Enguri River in Georgia and currently it is the world's second highest concrete arch dam with a height of 271.5 meters. It is located north of the town Jvari, and it is part of the Enguri hydroelectric power station which is partially located in Abkhazia. The dam is 650 meters long concrete double curvature arch dam has an estimated volume of 3,880,000 cubic meters of concrete poured into it, with a storage capacity of 1.1 billion cubic meters of water in its reservoir, refer to Figure 2 [5].

Maintenance of the dam and its power plant was neglected due to shortage of funds and the dam was in dilapidated conditions due to such negligence. Lack of maintenance works resulted in early aging signs within the dam and the power station. One of the power plant’s five units was shut for over 15 years and the other four were not able to work at their full capacity. At the same time, the dam’s structural stability and working conditions had deteriorated, with galleries inside the dam frequently flooding.

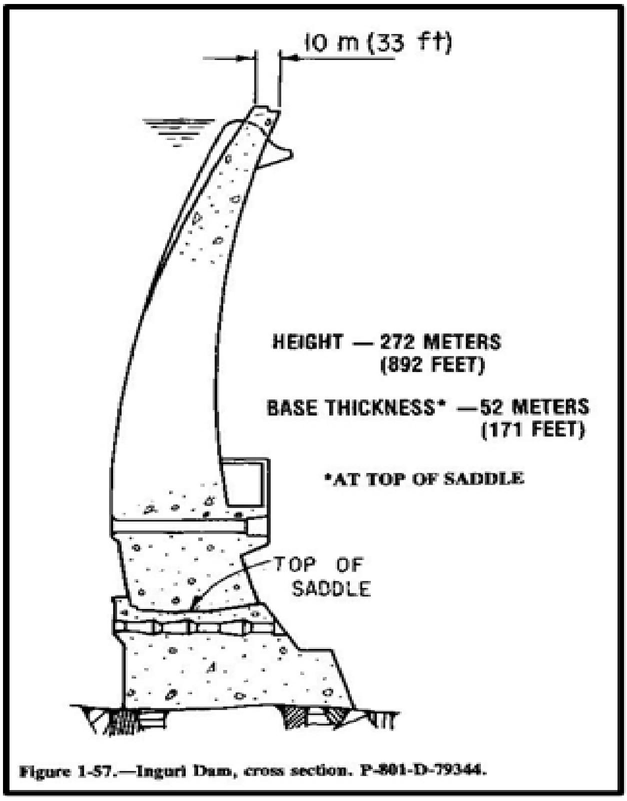

10 m (33 ft)
HEIGHT — 272 METERS (892 FEET)
BASE THICKNESS* —52 METERS (171 FEET)
*AT TOP OF SADDLE
TOP OF SADDLE
Figure 1-57.—Inguri Dam, cross section. P-801-D-79344.

Figure 2: Inguri Dam, cross section.

International experts identified the risks that would compromise the future of the dam back in 1994, and safety concerns were confirmed. In the years following the collapse of the USSR. Lack of maintenance of the dam resulted in a potentially dangerous situation. Close collaboration between the United Nations Educational Scientific and Cultural Organization (UNESCO) and the authorities in Georgia, which had raised concerns over the dam safety and called for technical and financial assistance, materialized in securing the required funds. The operations needed to remedy the situation were too costly for Georgia to bear alone. Remediation and renovation works where only possible after the involvement of the (UNESCO) which helped in enlisting the participation of the European Bank for Reconstruction and Development (EBRD) in 1998.These works covered the hydropower plant, civil engineering works on the structure as well as the upgrades of the generator units with financing totaling €58 million. The project was co-financed by the European Investment Bank (EIB) with €20 million. Moreover, the European Union provided grants for €9.4 million and additional €5 million were granted through its Neighborhood Investment Facility (NIF). In addition, grant funds from the Swiss government were used to finance engineering consultants at the design phase of the project [6] and [7]. More improvements to dam safety were needed in later years and the help of Russia was sought and agreement was reached between the two governments in 2003 [8].

Dams are expensive structures to fix. According to the US Association of State Dam Safety Officials, it would cost about $54 billion to rehabilitate all of the dams in the USA in need of repairs. The money would need to come from myriad sources, being, local, state, and federal coffers as well as thousands of private dam owners [9].

It is not surprising, therefore, to find even in rich countries like the United States that 95% of the of older dams lacked the maintenance needed to guarantee operational integrity and prevent failure. These aging dams represented a potential hazard to downstream communities which in most cases are unaware of the hazards posed by such dams [10].

The European countries with the largest installed hydropower capacity are Norway, France, Italy, Spain, Sweden, Switzerland, and Germany, maintaining or upgrading the existing infrastructure is an important focus throughout Europe. The emphasis in Western Europe is retrofitting hydropower plants with modern equipment and upgrading capacities of plants. In Eastern Europe, the focus is rehabilitating ageing plants that deteriorated during the Soviet era. Apart from the investment and production costs, the other principal cost element is operation and maintenance (O&M), including repairs and insurance, which can account from 1.5-5% of investment costs annually. Both the production and investment costs differ considerably depending on the head height of the plant [11].

Getting the required appropriations for dam upgrading is a long and complicated processes in USA as in all the other countries. For Federal Dam Rehabilitation; after dam safety deficiencies have been identified, rehabilitation activities should be undertaken. However, most federal agencies do not have funding available to immediately undertake all non-urgent repairs. Rather, they generally prioritize their rehabilitation needs, based on various forms of risk assessment, and schedule these activities in conjunction with the budget process.

At some agencies, dam rehabilitation needs must compete for funding with other construction projects.

As for the Rehabilitating of Nonfederal Dams, a task committee of the Association of dams in 2002 estimated that $36.2 billion was needed to rehabilitate these dams and that $10.1 billion was needed by 2014 for repairs to “the nation’s most critical dams. In the last update issued in 2019 it was estimated that $4.20 billion is needed to rehabilitate all federally owned dams with $2.93 billion of this attributed to the federally owned high hazard dams. Since 2004, an ASDSO task group has tracked dam rehabilitation cost for non- federal up to 2019 with potential sources of funding from dams as follows:

Table 1: Funding needs of non-Federal dams in USA [12].

Year	Funding needs, non-federal dams ($ Billion)	Funding needs, non-federal High Hazard dams ($ Billion)		
2003	**34**	**10,1**		
2009	**51,46**	**Public**	**Private**	**Total**
		8,7	**7,3**	**16,0**
2012	**53,69**	**Public**	**Private**	**Total**
		11,2	**7,0**	**11,82**
2016	**60,7**	**Total**	**18.71**	
2019	**65,89**	**20,24**		

As dams age and intensive developments continues in the floodplains below them, the structural integrity of such infrastructure becomes more significant public safety issue. Moreover, in view of recent climate change impacts, dams' planned capacity to withstand floods has come under increased scrutiny. However, it is unclear to what extent there will be a widespread reevaluation of flood and earthquake ratings at high-hazard dams. Such an evaluation could raise additional policy questions. For example:

i. What criteria should be used to determine whether current risks are acceptable.
ii. If risks are not acceptable, should the dam be improved by introducing changes to its design or may be changes in the downstream be undertaken, or even the option of removing the dam may serve best.
iii. Who should pay?

Regardless of whether dams were constructed to withstand an earthquake or flood of "appropriate" magnitude, they may have age-related deficiencies that need to be corrected to maintain current levels of safety against such events. Therefore, it is likely that appropriations for safety inspections and rehabilitation activities will continue and may increase.

In summary, when it comes to dam integrity, aging is very important matter to be concerned about. In many aging dams around the world structural components, such as; waterways, filters, drainages, and hydro-mechanical equipment are degrading. The process is accelerated by deposition of salts, oxidation and corrosion by chemical runoff or abrasion and potential cavitation of these waterways. Moreover, very old dams were designed and constructed by outdated engineering practices and hydrological and earthquake seismic events data which make them more vulnerable to failure. For many very old dams, the benefits to the public of removing them outweigh the costs of continued operation. In light of aging infrastructures, it is appropriate to evaluate individual dams to determine whether their ongoing economic and social costs justifies the services they provide.

3. World views on decommissioning and removal of old dams

The opinion on world dams' future has been sharply split between those countries which have satisfied their needs of water and power and have no more suitable sites to build new dams, and the other countries which have still large potentials to be taped by new dams and such dams are vital for their future development. In the more advanced countries new investments are preferably put in wind energy or solar energy sectors and this competes with rehabilitation and upgrading of their old dams. less developed countries are over burdened by the costs of loans for building their dams and the increasing costs of upgrading these dams.

The construction of large dams reached its peak in the 1970s in Europe and North America. Today most activities in these regions are focused on the management of existing dams, including rehabilitation, renovation and optimizing the operation of dams for multiple functions. An estimated 1700 large dams have been under construction in other parts of the world in the years leading to the end of last century. Of this total, 40% are reportedly being built in India.

According to the report of the World Commission on Dams (WCD-2000), management and operation practices of dams must adapt continuously to changing circumstances over the project's life and must address outstanding social issues. And for this end comprehensive post-project monitoring and evaluation process and a system of longer-term periodic reviews of the performance, benefits and impacts for all existing large dams are necessary. Programs to restore, improve and optimize benefits from existing large dams should be identified and implemented. Options to rehabilitate, modernize, and upgrade equipment and facilities; optimize reservoir operations; and introduce non-structural measures to improve the efficiency of delivery and use of services have to be addressed [13].

The other issues facing dam owners in the world nowadays are the outstanding social impacts associated with existing old dams such as the increasing safety hazards of such dams to communities and/or the increasing deterioration of the environment and ecosystems caused by these dams. In many cases dams' owners had concluded operating agreements with the regulating authorities such that their licenses expire after certain periods. Relicensing conditions of these dams requires generally that major improvements be either made, or decommissioning must be faced. ICOLD in its review of the WCD report endorsed its findings regarding the questions related to the physical sustainability of large dams and their benefits which confirmed that:

i. Ensuring the safety of dams will require increasing attention and investment as dams age, maintenance costs rise and climate change possibly alters the hydrological regime used as a basis for the design of dam spillways.
ii. Sedimentation and the consequent long term loss of storage of old dams is a serious concern, and the effects will be particularly felt by basins with high geological or human induced erosion rates, dams in the lower reaches of rivers and dams with smaller storage volumes.

Experience in North America and in Europe shows that decommissioning of dams has enabled the restoration of fisheries and riverine ecological processes. However, dam removals without proper studies and mitigation actions cause public concerns and environmental problems. These include negative impacts on downstream aquatic life due to a sudden flush of the sediments accumulated in the reservoir. Where there has been industrial or mining activity upstream, these sediments may be contaminated with toxic substances.

While decommissioning efforts in the United States and France have received public support thus far, there may be local opposition where changes in the flow and water levels affect services previously provided by the dam, or where development has taken place around the reservoir and downstream. There is comparatively little experience with the removal of larger dams. The bigger the dam, the more problems decommissioning or removal are likely to face, and the more expensive they are likely to be. More studies are needed to address the costs, benefits, and impacts of decommissioning as dams age and choices must be made between refurbishing and decommissioning.

4. Aging Dams and Ecosystem Restoration

The question of ecosystem restoration has been undertaken in late years in a range of countries where evolving national legislation has required higher standards of environmental performance. In the United States and France, dams have been decommissioned to restore key environmental values, often related to migratory fish (salmon), and often as a condition of project relicensing when there has been serious safety hazards. Substantial ecological degradation can be attributed to the increasing number of dams built on rivers and often on the same river creating obstacles to fish migration and have drastically altered river flow regimes. Moreover, many dams have been in place for 50 years or more, and increasing number of them are now approaching or exceeding their originally intended design life and will require very large investments to reach acceptable levels of safety and function.

Decommissioning of dams and restoration of ecosystems involve costly works, therefore, have often been limited in scale, and its effectiveness is frequently unclear. Nevertheless, there is growing demand, political will, and funding for restoring degraded ecosystems. A total of 467 dams have been removed to date in the United States, 28 of these are large dams higher than 15 meters. Reasons for removal have included safety concerns, the restoration of riverine fisheries, financial considerations, or removal of unauthorized structures.

One example of a removal cases is the Grangeville dam on Clearwater Creek River, Idaho. Grangeville (Harpster) Dam was built on the South Fork of this river in 1910; it housed a 10MW power plant. The removal was motivated by excessive sedimentation in the reservoir and blockage of migratory fish following the collapse of the fish pass in 1949. The dam was removed in 1963, and the river washed out the accumulated sediment within six months with no recorded downstream effects. The demolishing of this dam restored the river's populations of Chinook salmon and steelhead trout and restored access for fish runs to 67km of main stem river and over 160km of tributary habitat in the upper reaches of the Clearwater River. It also allowed members of the Nez Perce Indian tribe to regain a traditional fishery long denied them despite the provisions of the 1855 treaty with the United States. Today, the entire South Fork and its headwater tributaries are free flowing and unobstructed by dams or major diversions [14] and [15]. The restoration of ecosystem involving removal of any dam is not an easy task and it can be complicated for the following reasons:

i. Ecosystem impacts of decommissioning can be complex and site-specific. One major issue in dam decommissioning is what to do with possibly polluted sediment accumulated behind the dam. The fate of this sediment when the dam is removed is frequently a major obstacle to restoration.
ii. Current large dam designs are often not sufficiently flexible to allow for changed operating regimes to meet environmental (or other) goals. Global experience shows that these long-lived structures may be called on to operate differently in the future than in the past as society’s needs and values evolve and as other dams are added in the catchment area.
iii. In some cases, the dam design is completed before the environmental flow needs are determined, and cannot accommodate water releases of the required quantity and quality. Five dams on the Colorado River have now been retroactively fitted with variable level offtakes to draw off surface water, increase the temperature of the downstream river, and satisfy the needs of native fish.
iv. Decommissioning can be costly when subjected to cos/ benefit analysis, but sometimes enough justifications are presented to show lucrative advantage. This was made clear in one specific case by an editorial report titled “Dollars, Sense & Salmon” which addressed such a case in 1997. The document was researched by Statesman reporter Rocky Barker and written by editorial writer Susan Whaley. The team dug deep into the numbers and invited experts to discuss the benefits and shortcomings of the four lower Snake River dams and an argument for breaching these dams was strongly advocated. The Statesman cited a net annual benefit of $183 million should the lower Snake River dams be removed.

In 2016, a full 19 years after the Statesman published the report, a historic opportunity came when in May a federal judge in Portland ordered the government to write a new environmental study that weighs a range of alternatives, including lower Snake River dam’s removal [16].

5. Removal of aging dams

A decision of removing an old dam may be based on two major considerations, which are,

i. Public Safety concerns.
ii. Economics.

If the risks involved in keeping the dam are too great to the downstream communities in case of failure, then it may be best to remove it. Sometimes, the problem can be solved by partial removal of the dam rather than by full removal. This may be done by lowering the maximum operation water level of the reservoir in order to permanently reduce the loads on the dam and the potential downstream consequences in the event of dam failure. In such cases the extra costs of total removal can be avoided. A controlled breach of an embankment dam by means of a notch requires engineering analysis to assure proper sizing, shaping, and armoring to prevent instable conditions and future flooding. Lowering the maximum operation water level may also be accomplished by non-structural methods, such as permanently opening (or removing) gates from the spillway and/or outlet works. Full or partial removal of any type of dam requires the consideration of wide variety of technical, environmental, social, political, and economic issues [17] and [18].

The question of costs involved may be looked at by a cost-benefit approach. The benefits which can still be derived by keeping the dam such as flood control, agriculture, power generation, and recreation will have to be compared with the economic costs and social costs that have to be borne by both owners and society.

The long-term maintenance costs and costs of removal added up together make up the owners share in this evaluation. Social costs may be evaluated by assessing the opportunity cost of hindering the free migration of fish and other species, degraded water quality and negative impacts on the ecosystems and free flowing rivers in addition to cultural values.

In the United States agencies like the U.S. Army Corps of Engineers (USACE) and the Federal Energy Regulatory Commission (FERC) have established processes to evaluate benefits and costs as a part of various agency programs.

Licensing decisions at (FERC), for example, consider multiple management scenarios when evaluating whether to issue new or renew existing dam license. The management scenarios may require dam owners to allow greater water flow through the dam, install infrastructure to allow migratory fish to pass upstream, or make safety upgrades. Often the options include a dam removal scenario. These evaluations also include impact assessments that evaluate the benefits and costs to the many parties affected by each management alternative.

The USACE undertakes similar analyses when evaluating its dams and other river restoration programs. Some dam owners have found that removing a dam is more appropriate than leaving it in place after comparing benefits and costs of addressing the needs of concerned parties and meeting state and federal regulatory requirements. Since 1912, more than 1,300 dams have been removed across the U.S., and 62 dams were removed in 2015 [19].

It can be said however at least for the present and for the foreseen future of the economic environment, that the rising costs of operating and repairing dams, and the improved awareness of the economic and social benefits of removing them, has shifted the balance sheet for some dams towards their removal.

One report, published in 2016 by “Head Water Economics Group”, lists many dam removals in USA. The case studies, benefits of dam removal, and alternatives considered are listed as shown in Table 2 [20]. The “Headwater Economic Group” presents itself as an independent, nonprofit research group whose mission is to improve community development and land management decisions in the west of the USA.

Table 2: Case studies, benefits of dam removal, and alternatives considered.

Location	**Estimated Cost of Removal $ (2016)**	**Estimated Benefits of Removal $ (2016)**	**Alternatives to Dam Removal**
Whittenton Pond Dam, Mill River, Massachusetts	$447,000: 99 percent paid by state and federal partners, $1.5 million for avoided emergency response	$1,5 million for avoided emergency response. Increased numbers of two vulnerable species: American eel and river herring. Property values projected to increase due to lower flooding risk.	Rebuilding was necessary due to disrepair and safety hazard, cost estimated at $1.9 million
Elwha and Glines Canyon Dam, Elwha River, Washington	$324.7 million	$5,3 million annually from increased commercial fishing. Cultural and public safety benefits to the Lower Elwha Klallam Tribe, downstream from the dams. $33 million in personal income and 760 new jobs associated with dam removal $43,8 million and 446 new jobs from 500,000 more visitor days annually. $5,3 billion worth of improved wellbeing for the American public.	Not available

Edwards Dam, Kennebec River, Maine	$10,9 million	$2.5-$38.2 million for improved recreational fishing quality $397,000-$2.7 million for improved river recreation quality. Property values closest to the former dam site increased Electricity produced by Edwards Dam cost 4-5 times the market rate. Water quality prior to dam removal did not meet minimum standards; afterward it could support all native fish Alewife population increased 60-fold, and they now are used commercially for bait. Quality of life in Augusta has improved due to new connection to the river.	$14.9 million to install fish passages and conduct environmental mediation
Condit Dam, White Salmon River, Washington	$24,8 million	Cultural benefits for the Yakama Nation from returned salmon and lamprey, including sustenance fishing. Expanded spawning grounds for recreationally and commercially important fish: 12 miles for salmon and 33 miles for steelhead. Increased populations of five fish species listed under the Endangered Species Act. 30,000 additional whitewater boaters annually.	$52.4 million for fish passages, plus $3.9 million annually in higher electricity costs
Great Works and Veazie Dams, Penobscot River, Maine	$65 million	76 jobs and $3,6 million in economic impact from dam removal. Access re-opened for 1,000 miles of habitat for 11 depleted historic fisheries. Cultural and sustenance fishing benefits for the Penobscot Indian Nation. New area spending by whitewater boaters, including several events.	Fish passage facilities were insufficient to restore fisheries
Small Dams: Hyde Pond Dam, Whiteford Brook, Connecticut	$1,1 million	Avoided public safety hazards from catastrophic failure and upstream flooding. Four miles of stream habitat opened to fish species including American eel, a vulnerable species.	Dam would have to be rebuilt to meet safety standards. Dam owner would have been responsible for full cost of rebuilding dam

Small Dams: Bartlett Pond Dam, Wekepeke Brook, Massachusetts	$325,000	Avoided public safety and infrastructure hazards from catastrophic failure and upstream flooding. Eighteen miles of stream habitat opened for brook trout and other species.	$671,000 for repairs
White Rock Dam, Pawcatuck River, Connecticut and Rhode Island	$800,000	Avoided public safety and infrastructure hazards from catastrophic failure and upstream flooding. Twenty-five miles of river habitat opened to fish species.	Dam would have to be rebuilt to meet safety standards. Dam owner would have been responsible for full cost of rebuilding dam

One very important issue which has to be studied carefully if dam's decommissioning is considered is siltation. This is often a critical issue when the reservoir accumulated sediment volume is very much greater than the mean annual sediment load. In such case sediment-related impacts following dam removal can occur in the reservoir itself by activating fast rate erosion of the catchment and in the river channel upstream, and downstream from the project site by affecting installations and clogging water intakes and causing high turbidity depending on the local conditions and the removal methods and rates, the degree of impact can range from negligible to significant. For example, removing a small diversion dam that had trapped only a small amount of sediment would not have much impact on the downstream river channel. Furthermore, if the upper portion of a dam is removed in such a way that very little of the existing reservoir sediment are released downstream, the impacts to the downstream river channel would be related only to the future passage of sediment from the upstream river channel through the remaining reservoir. However, if dam removal results in a large quantity of sediment being released downstream, then the impacts to both the upstream and downstream channels could be significant. Case by case studies must be made, for investigating such things as erosion of the river channel both upstream and downstream of the removed dam site, water quality with respect to turbidity, and impacts on both aquatic plant ecosystem and biodiversity [21].

6. Dam removal of old dams: case studies

Three cases are selected in the following to illustrate different dam decommissioning cases from three different countries, namely; France; Germany; and USA.

6.1 The Vezins and La Roche Qui Boit hydroelectric dams (France)

These two concrete dams were located in the (Normandy, France), had been operating since the 1920s and 1930s and they were showing signs of aging presenting safety and profitability problems and loss of storage by sedimentation.

As early as 2005 the owner of the dams had been positioning these dams for removal in order to restore the Sélune fish migratory axis. And in 2009, based on a report by Électricité de France (EDF) in 2007, and the lack of technical possibilities to rearrange the dams to ensure fish and sediment continuity, the State decided not to renew the licenses of the two dams and to begin the dismantling process. The removal of the Vezins and La-Roche-qui-Boit dams was definitively decided in 2017.

Since 2009 and in order to give an operational follow-up to this decision, the Minister of State and the Secretary of State for Ecology have instructed the Prefect of the Department (la Manche) to launch the operations required for the successful dismantling of the two dams and re-naturalization of the river.

Considering the clear desire to implement this project in terms of the environment and sustainable development, the State has launched numerous studies aimed at detailing technical choices. By defining the parameters such as re-naturalization, sediment management, waste management associated with decommissioning, fish management, etc. with the goal of achieving a zero state of the river, essential elements on which to assess the biological benefits, the works will take into account the effect of the removal on local activities and especially the impact on tourism on the surrounding villages. The state was very concerned about the potential risks of the operation and has established:

- A flood risk study to analyze the current effect of the two dams on downstream flood flows.
- A study on the state of polluted sediments present on the site, including A characterization of the sediments and a site management plan.
- A study of the release capacity of contaminants present in the sediments of the reservoir.
- From 2012 a scientific program was conducted to monitor the dam removal operation. It was organized around four interrelated themes: Landscapes, Inhabitants, Uses, Landscape, and Agriculture, Fluvial Dynamics and Aquatic Biocenosis.

As part of the socio-geographical component of this program, a team of geographers and sociologists worked on the transformations affecting the landscapes but also the uses and representations of the valley. This work proposes an inventory of fixtures, a history and a follow-up of the landscapes and uses of the valley of Sélune [22].

Figures 3 and 4, are photos of the Vezins and La Roche Qui Boit dams before removal [23]. Figures 5 and 6 show Vezins dam at various stages of demolition [23], [24].

The removal of both dams was part of an ambitious program, unprecedented in Europe, to restore ecological continuity between the terrestrial and marine habitats connected by river basins [25].

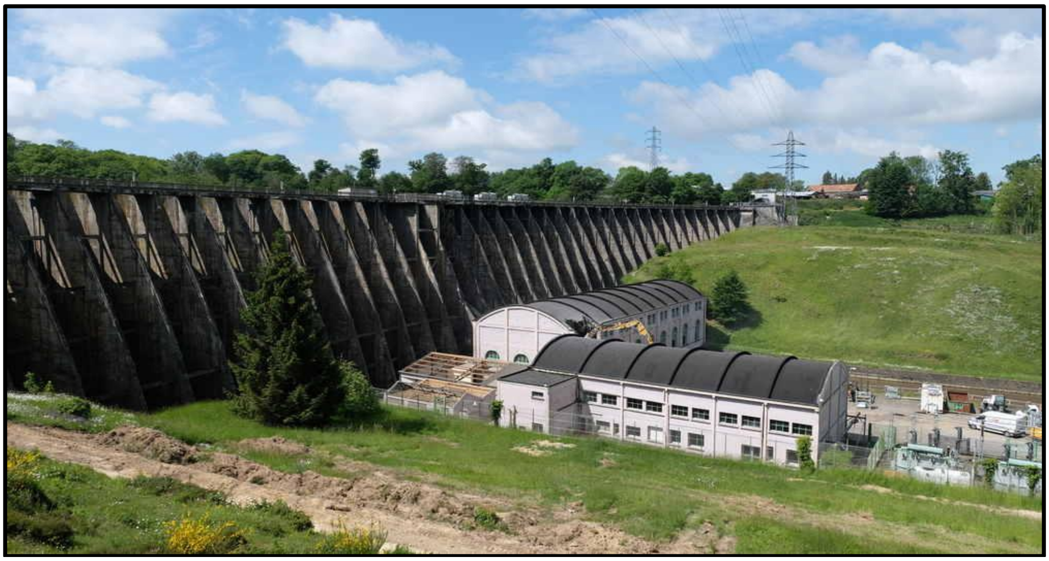


Figure 3: Vezins dam before removal [23].

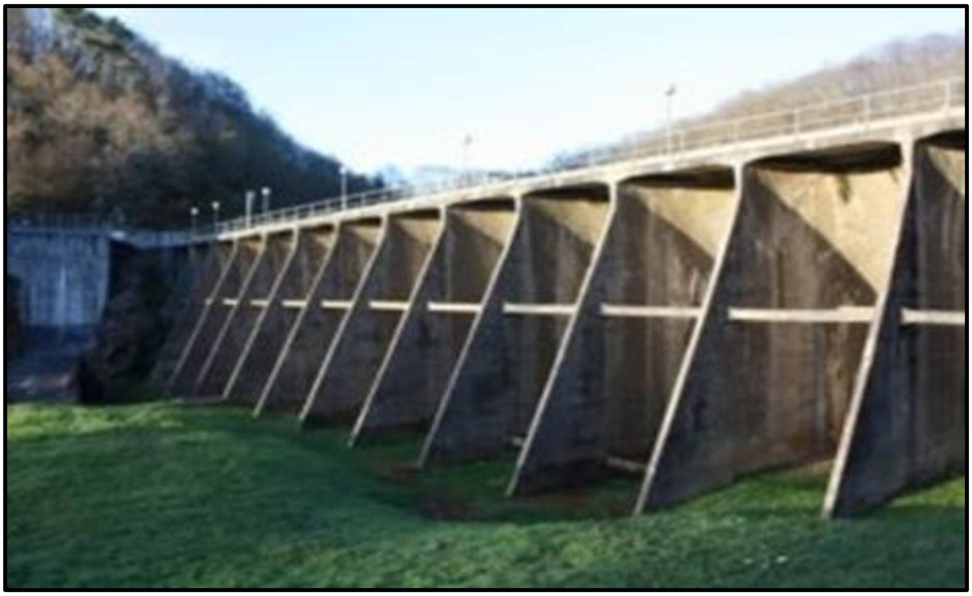


Figure 4: La Roche Qui Boit before removal [23].

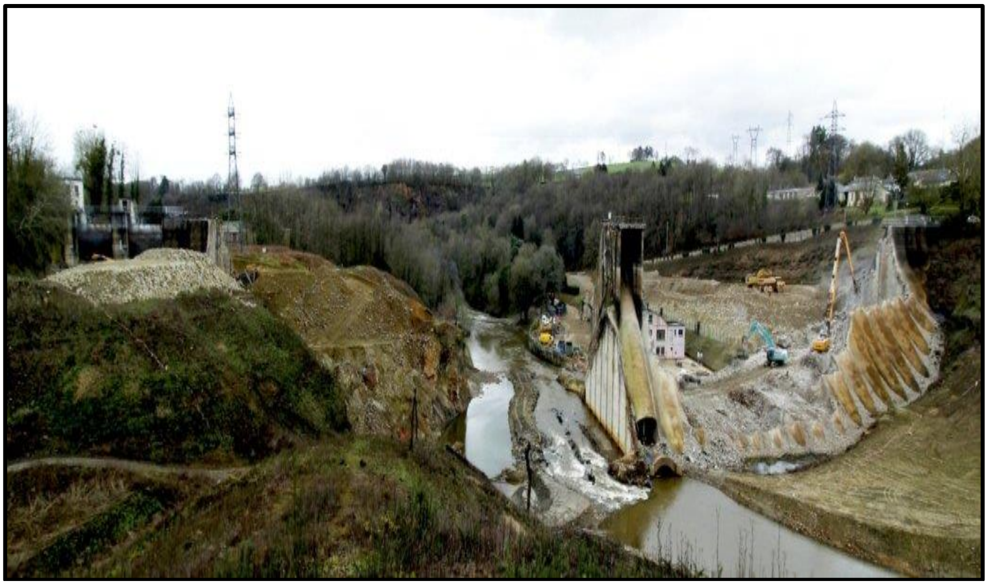


Figure 5: Vezins dam at an advanced stage of demolition [23].

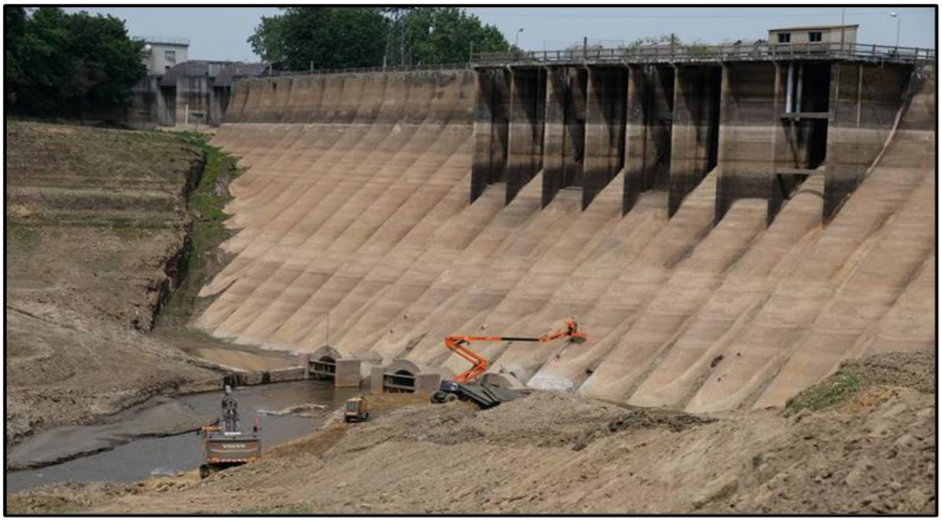


Figure 6: Vezins dam at an early stage of demolition [24].

Other dam removal cases are also mentioned in the literature. Such cases are the case of Saint-Etienne du Vigan Dam (built in 1895, dismantled in 1998) and the Poutès Dam built in 1941. Both dams were located on The Allier River, main tributary of the river Loire in France [26].

6.2 The Krebsbach Dam, Thuringia (Germany)

The Krebsbach Dam was built in 1962 in East Germany on a small stream which served industrial water supply for the mining of uranium in the area. The 18.5m dam was high rock fill dam with an inner core of clay. Its length was 186 m, its crest width was 5.1m and it retained 320,000m^3 of water. In 1985, after mining had stopped the dam was abandoned. By the 1990's, stability problems had worsened as the dam was suffering from structural damage. New laws dealing with security and safety risks associated with flooding forced the owners into action. Comprehensive restoration and continuing maintenance of the dam would have been expensive in order to meet the new regulations which led the owners to decide in 1998 to demolish the dam completely. The aim was to remove the dam and all the technical equipment associated with uranium processing and to restore the continuity of water flow in the stream. But before removal could start, planning approval had to be sought in line with Germany's Water Resources Act. By 2001, planning for removal begun. The planning procedure had to break new ground as there was no precedent in Germany for the demolition of a large dam. An environmental impact study was carried out, a landscape conservation plan was written and stakeholders, including the public and those concerned about effects on the environment, were asked for their opinions. The planners received fifty-three responses which were discussed in detail at a public meeting in May 2003. The main concern of local residents was that there would be a greater risk of flooding to the downstream stretches after the dam had gone.

The environmental impact assessment and landscape conservation plan were approved in 2005 and preparations for demolition started in 2007. Although sedimentation was relatively low and only averaged 35cm in the reservoir but sediment depth was as large as 2.5m close to the dam. A sedimentation pool was created directly downstream of the dam structure to minimize the transport of sediment by related discharge into the stream's lower reaches during demolition of the dam. All fish, amphibians and shellfish were relocated from the whole length of the stream (as it was assumed that water quality would deteriorate) and then the reservoir was drained, and the dam was completely demolished , refer to Figure 7 [27], [28].

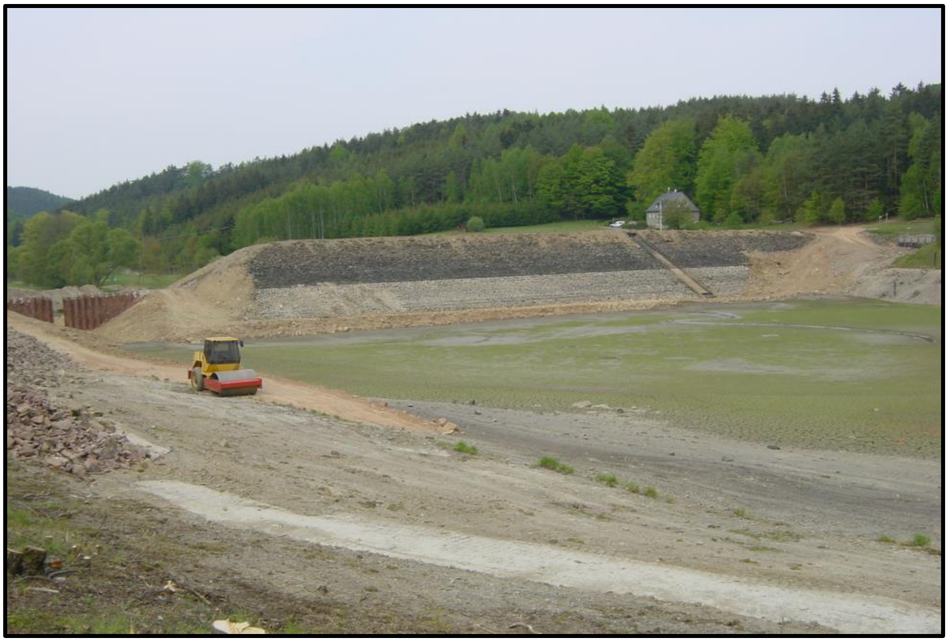


Figure 7: Dam removal in progress in 2007.

The actual removal process was fully described with lessons learned after eleven years in the International Seminar on Dams Removals that was held by Karlstad University (KAU) in Sweden on 24-26 September 2018 [29].

6.3 Elwaha and Glines Canyon Dams, Washington (USA)

Removal of the Elwha and Glines Canyon Dams from the Elwha River in Washington in 2012 was the largest dam removal and river restoration project in the United States to date. Before these two dams were built, the river supported ten runs of salmon and trout, including all five Pacific salmon species. Removing these two dams was the only way to restore these fish runs. This project was a unique opportunity for fishery restoration because the upper section of its watershed lies entirely in Olympic National Park, increasing the chances of successful recovery. The locations of the two dams are shown in the map of Figure 8.

Completed in 1913, the Elwha Dam was located five miles upstream from where the Elwha River empties into the Strait of Juan de Fuca. It was 105 feet high and had a 14.8MW generation capacity. The Glines Canyon Dam was completed in 1927 and was 13 miles from the Strait of Juan de Fuca. It was 210 feet high and had 13.3MW generation capacity. Both dams were used to generate hydroelectric power for nearby paper and lumber mills. Both dams failed safety inspection in 1978 which was followed by modelling study of flood hazard should the dams fail that highlighted potential harm to the communities downstream.

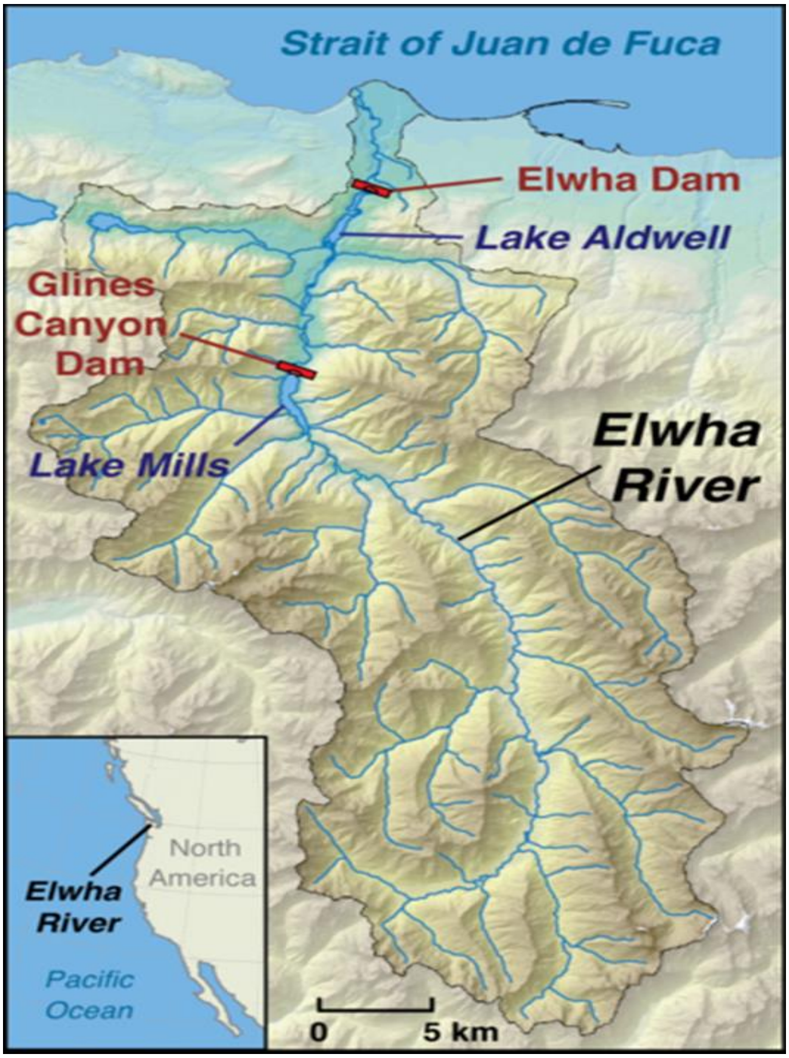


Figure 8: Location of Elwaha and Glines canyon dams on Elwha River [30].

An act of the Congress in 1992 revoked renewal license of the dams which was issued by FERC and called for restoration of the river. This act stipulated the following:

i. Removal of FERC's authority to license the Elwha Project.
ii. Required federal studies to research alternatives for full restoration of the Elwha River ecosystem and migratory fisheries.
iii. Authorized the Secretary of Interior to purchase and acquire both the Elwha and Glines Canyon Dams for a fixed price and then implement necessary actions to meet full restoration objectives. Thereafter the two dams were purchased from the owner in 2000 for $29.5 million.

Two environmental impact statements (EIS) concluded that neither leaving the dams intact nor installing fish passages would be sufficient to restore the fisheries. Moreover, they both posed Safety risk to the public [31]. As a result, the Elwha and

Glines Canyon Dams were removed in 2012. The total cost of purchasing and removing the dams and hydropower facilities, and conducting river restoration activities was $324.7 million. Costs and benefits of the restoration of the project are given in Table 2 shown already. Photos showing the Elwaha Dam before and after removal are presented in Figures 9 and 10. The stages of removal operation can also be viewed in an interesting video film posted on the following link; https://youtu.be/m96VcCF4Ess

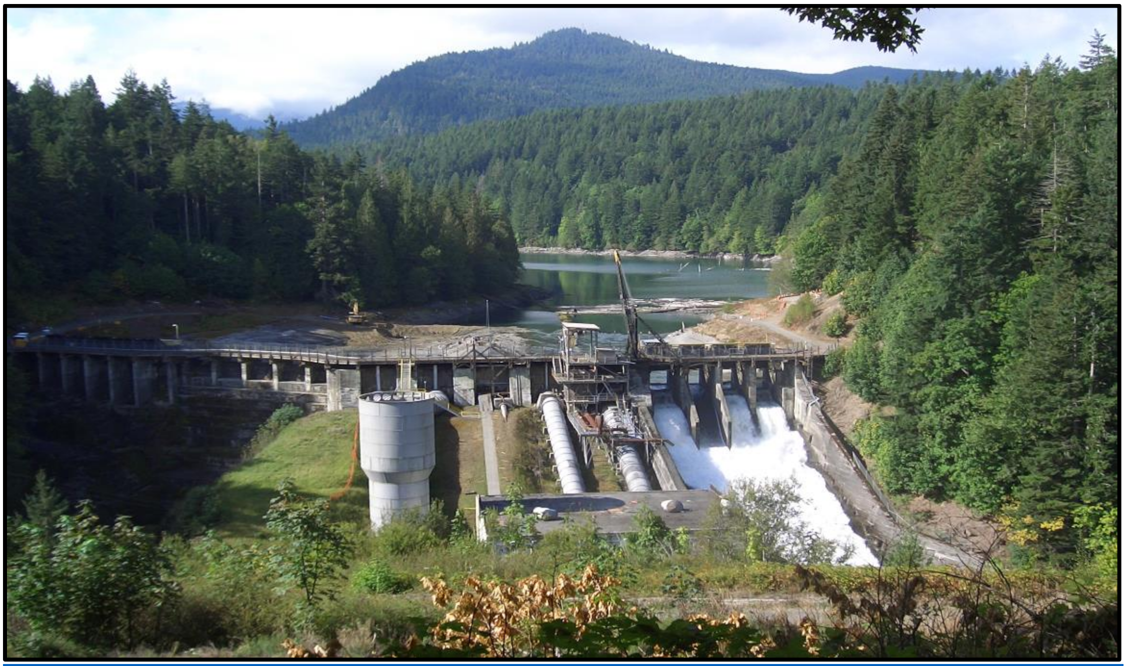


Figure 9: View of Elwha Dam before removal.

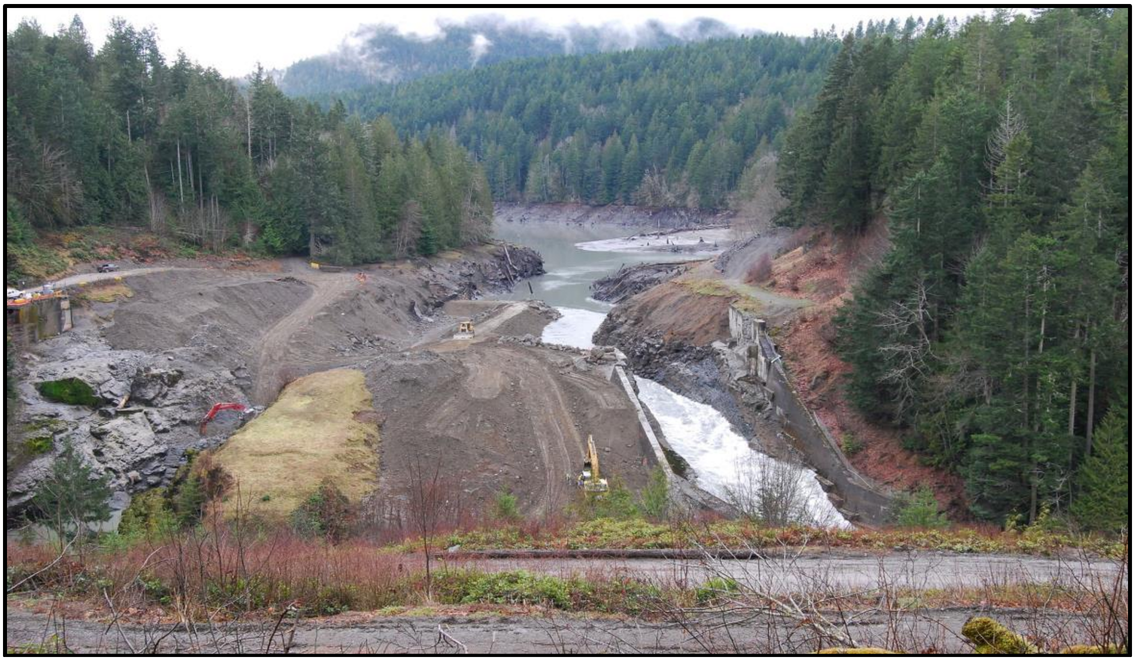


Figure 10: View of Elwha Dam after removal.

7. Summary points and conclusions

From the proceedings it is clear that aging dams present serious problems of safety risks and negative impacts on the environments of rivers, in addition to diminishing benefits and soaring costs of repair and upgrading. As conclusions the following may be drawn:

1. Old dams of 50 years of age or more may not meet today’s requirements of the accumulated hydrological and seismic activity records collected during these past years, nor they satisfy current design criteria of design which have evolved after the construction of such dams. Moreover, the current climate change impacts on the hydrological cycle have increased the vulnerability of these dams.
2. Case studies have shown that cost of upgrading and proper maintenance of old dams, increases with age against a decreasing benefits. Such decreasing may result from shrinkage of storage due sedimentation of reservoir or restricting operation water levels to limit the hydrologic impacts of un-reckoned floods or reducing the load on the dam itself due to stability problems.
3. Budgeting limitations can exasperate safety problems of old dams which normally need more attention and maintenance and suffer from diminishing returns at the same time. Safety issues therefore are leading issue in the decommissioning of old dams or remove them altogether.
4. The development of environmental protection legislation in many advanced countries has given the question of ecosystem restoration by removing old

dams more prominence. Higher standards of environmental performance required from old dams has led to many decisions for decommissioning of such dams. Key environmental values on which decommissioning or removal of old dams are based are often related to migratory fish (salmon) and ecological degradation of the rivers due to increasing number of dams built on these rivers which drastically alter river flow regimes.

5. Removing an old dam is not an easy task. It requires careful pre-planning, engineering and environmental studies, and design work. Without this, negative consequences of various types and magnitudes can happen. Uncontrolled release of water leads to increased flood hazards, washing large loads of the accumulated sediment can clog and overload water supply schemes, degrades water quality by higher turbidity. In the long run scouring of the downstream reach may threaten the foundations of bridges, jetties and other structures built on the river such as bridges and jetties by eroding them requiring repair works and additional costs. Moreover, scouring of the upper reaches of the river channel bed is another possible outcome which increases erosion of the catchment and impacts the stability of aquatic ecosystem and biodiversity there.
6. Thousands of small dams and few moderate size dams have been removed already mostly in USA and some European countries. There is comparatively little experience with the removal of very larger dams. The bigger the dam, the more problems decommissioning or removal are likely to face, and the more expensive they are likely to be. More studies are needed to address the costs, benefits, and impacts of decommissioning as such dams age and choices must be made between refurbishing and decommissioning.
7. Preparations for old dams' removal can take very long time to complete the necessary studies and to get the required licenses from the legislative and administrative authorities and finally getting the required allocations. Each case has to be the subject of benefit- cost analysis. Admitting at the same time that obtaining values of benefits and social costs related to environmental questions is not an easy task and may be biased due to community and NGO pressures. Keeping old dams, however, must be scrutinized carefully taking into consideration costs of upgrading and maintenance with the benefits still available, but keeping in mind that safety and reduction of risks on downstream communities must come first.

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